DTIC
ELECTE
JAN 2 4 1992
S D
D

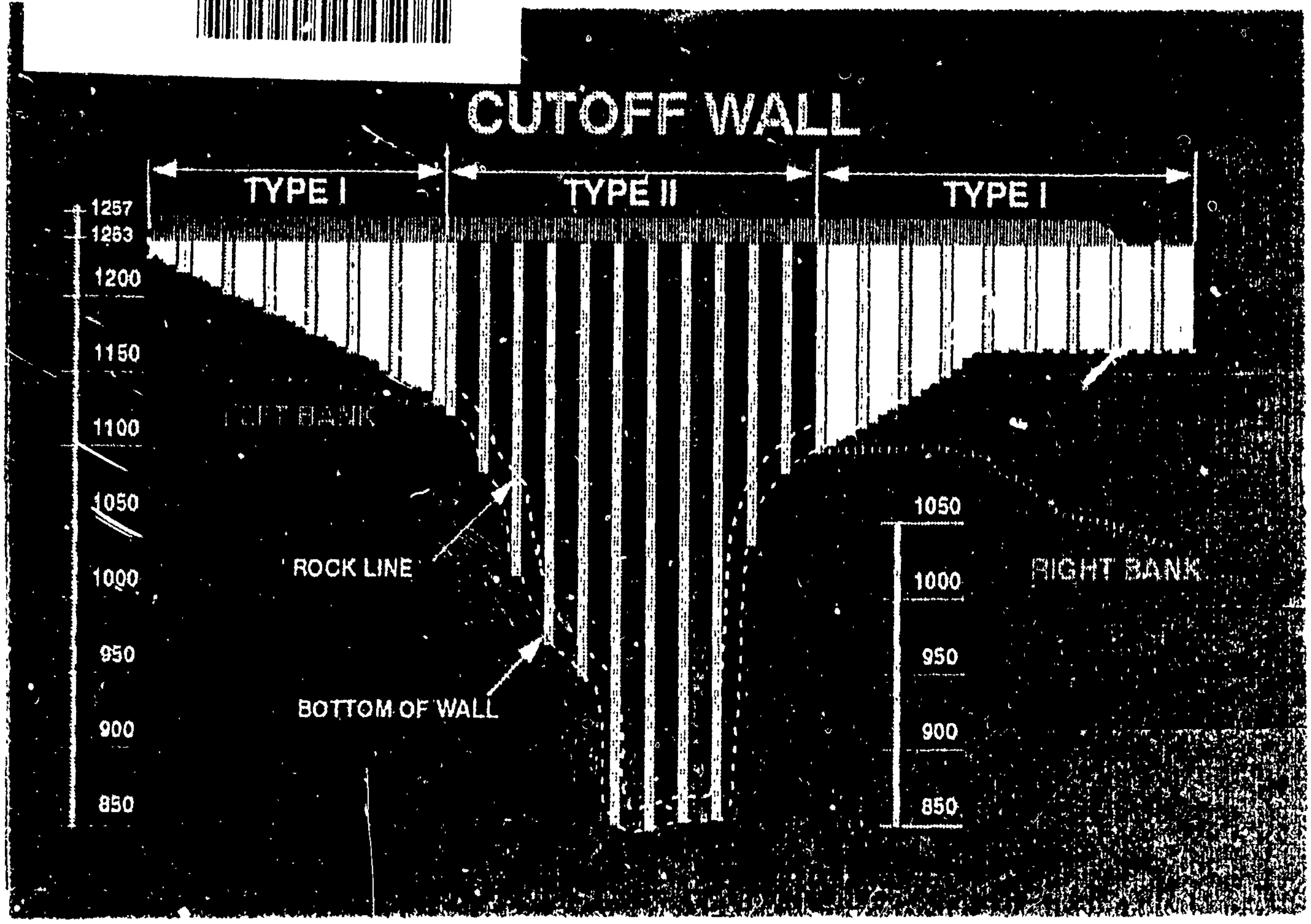
000033

1

US Army Corps
of Engineers
Seattle District

Construction Foundation Report

AD-A244 923



This document has been approved for public release and sale; its distribution is unlimited.

Mud Mountain Dam Seepage Control Cutoff Wall

White River, Washington

92-01911

92 1 23 069

August 1991

Accesion For
NTIS CRA&I
DTIC TAB
Unannounced
Justification
By per form 50
Dist
A-1

CONSTRUCTION FOUNDATION REPORT
SEEPAGE CONTROL WALL
CONTRACT NO. DACW67-88-C-0047

DTIC COPY INSPECTED 4

MUD MOUNTAIN DAM (LOOKING N.E.). SPILLWAY (FOREGROUND) WITH DAM AND DOWNSTREAM ACCESS ROAD (RIGHT CENTER). PROJECT AND RESIDENT OFFICES TO THE LEFT OF SPILLWAY.

MUD MOUNTAIN DAM

CONSTRUCTION FOUNDATION REPORT, AUGUST 1991

This report has been prepared by the following Seattle District personnel assigned to the Mud Mountain Resident Office:

NAME	SIGNATURE	DATE
Hiroshi Eto, P.E. Acting Resident Engineer	Hiroshi Eto	28 Aug 91
Matthew V. Satter, P.G. Resident Geologist	Matthew V Satter	28 Aug 91
Kenneth L. Forbes Certified Concrete Construction Inspector	Kenneth L. Forbes	28 Aug 91

TABLE OF CONTENTS

CONSTRUCTION FOUNDATION REPORT
SEEPAGE CONTROL WALL
CONTRACT NO. DACW67-88-C-0047

SECTIONS

SECTIONS CONTINUED

SECTIONS CONTINUED

PERTINENT DATA

1. General

Federal Identification Number	WA00300
Owner/Operator	U.S. Army Corps of Engineers, Seattle District
Date Constructed	1939-1942 and 1947-1948
Purpose	Flood Control
Downstream Hazard Potential	Category 1 (high)
Size Classification	Large

2. Location

County, State	King/Pierce, Washington
GLO Location	Sec 17, T19N, R7E, W.M.
USGS quadrangle	Enumclaw
Latitude	47° - 8.4'
Longitude	121° - 55.9'
Upstream from Mouth of White River	28 miles
Upstream from Mouth of Puyallup River	38 miles

3. Reservoir Data

Watershed	Upper White River
Drainage Area	400 square miles
PMF Outflow	245,000 c.f.s.
Capacity at Spillway Crest	106,000 acre-feet
Capacity at Pool Elevation of 1252 feet	147,500 acre-feet
Pool at PMF	1,252.2 feet

4. Dam

Type	Rockfill (Concrete cutoff wall in earth core)
Structural Height	432 feet
Hydraulic Height	360 feet
Crest Elevation	1,257 feet msl
Crest Length	810 feet
Width	
at Base	1,600 feet
at Crest	25.5 feet
Volume of Fill	2,300,000 cubic yards
Concrete in Project	87,000 cubic yards
Concrete in Cutoff Wall	17,000 cubic yards
Design Freeboard	4.8 feet

5. Spillway

Location	Right Abutment
Type	Concrete Free-Overflow Chute

Crest	1,215 feet
Elevation	315 feet
Width	1200 feet
Length	245,000 c.f.s.
Capacity at Pool Elevation 1,252.2 feet	

6. <u>Outlet Works</u>

9-Foot Tunnel	
Type	Concrete, Horseshoe
Location	Right bank
Length	1,800 feet
Intake Elevation	895 feet
Control	9-foot Radial Gate at Upstream End
Discharge at Pool Elev. 1,215 feet	5,200 c.f.s.[1]
23-Foot Tunnel	
Type	Concrete, Circular
Location	Right Bank
Length	1991.5 feet
Intake Elevation	970 feet msl
Control	Three Penstocks
Discharge at Pool Elev. 1,215 feet	12,400 c.f.s.[1]
Penstocks (in 23-foot tunnel)	
Number	Three
Length	867.5 feet
Diameter	8.5 feet
Regulating Valves	Three 8-foot Howell-Bunger valves
Discharge at Pool Elev. 1,215 feet	approx. 4,200 c.f.s. for each valve

[1] **Discharge for new intake structure:**

Total authorized flood control discharge:	**17,600 c.f.s.**
9' Tunnel discharge @ elev. 1,215' msl:	**4,600 c.f.s.**
23' Tunnel discharge @ elev. 1,215' msl:	**13,000 c.f.s.***

*** maximum possible discharge: 19,550 c.f.s.**

SEATTLE DISTRICT, US ARMY CORPS OF ENGINEERS

MUD MOUNTAIN DAM
REPORTS AND REFERENCES

Subject	Date
Construction Plans	—
Geological Report, Edwin T. Hodge	18 July 1938
Analysis of Design, Mud Mountain Dam	3 December 1938
Design of Mud Mountain Dam and Appurtenances	30 December 1938
Report on Soil Tests for Mud Mountain Dam	11 May 1939
TM 164-1, WES, Results of Soil Test on Materials from Proposed Mud Mountain Dam	20 December 1939
Contract Specifications for the Dam	1939
Test Fill Report for Mud Mountain Dam	—
Bonneville Hydraulic Laboratory, Model Study of the Spillway for Mud Mountain Dam	1942
WES Bulletin No. 14, Permaeability Characteristics of Mud Mountain Impre-vious Clay Materials	20 February 1942
Foundation Report	21 March 1942
Bonneville Hydraulic Laboratory, Model Study of the 23-Foot Outlet Tunnel for Mud Mountain Dam	15 July 1942
Prototype Testing	January 1945
Agenda for Consulting Board Meeting	28-29 July 1945
Analysis of Design—Embankment Design	1946
Analysis of Design—Design Other Than Embankment	1946
Analysis of Design—Original 9-Foot Gate, Modified 9-Foot Gate and Cable Way	1946
Master Recreation Plan	1946
Real Estate, Preliminary Planning Report—Civil Project—Tracts 14 and 15	4 March 1948

Subject	Date
Supplement to Agenda for Consulting Board Meeting	28-29 July 1948
Proceedings of Consulting Board Meeting	28-29 July 1948
Real Estate, Preliminary Planning Report—Civil Project—Easements for Radio Reporting Network, Puyallup River Basin, Mud Mountain Dam	4 May 1949
Report of Earthquake Damage	1949
Proposed Improvements—Right Bank	January 1950
Proposed Revisions—Outlet Works	January 1950
Real Estate Supplement to Preliminary Planning Report—Easements for Radio Reporting Network, Puyallup River Basin, Mud Mountain Dam	13 February 1950
Real Estate Supplement to Supplement dated 13 February 1950 (same title)	6 March 1952
Reservoir Regulation Manual	August 1954
Real Estate Planning Report—Civil Project—Mud Mountain Dam, Piezometer Stations	26 August 1954
Real Estate--Design Memorandum No. 1--Access Road and Bridge	11 February 1958
Design Memorandum (unnumbered)—Improved Access Upstream and Downstream Outlet Works Structures	December 1960
Design Memorandum Supplement No. 1—Improved Access to Upstream and Downstream Outlet Works Structures	December 1962
Design Memorandum No. 1A (no approval date)	--
Design Memorandum No. 1B—Reservoir Management and Public Use Development (Master Plan)	March 1964

Subject	Date
Report of Earthquake Damage	1965
Periodic Inspection and Continued Evaluation Report—Inspection of:	20 July 1967
Design Memorandum No. 1B Revised Master Plan	December 1968
Periodic Inspection and Continuing Evaluation Report—Inspection of:	29 April 1969
Periodic Inspection and Continuing Evaluation Report No. 3—Inspection of:	7 October 1971
Environmental Impact Statement—Mud Mountain Dam and Reservoir, White River, Washington	April 1972
Cableway Replacement Planning Report	May 1973
Periodic Inspection and Continuing Evaluation Report No. 4—Inspection of:	23 May 1973
Feasibility of Forest Management Determination Report	December 1973
Mud Mountain Dam, Interpretive Concept Plan	April 1974
USGS Water Resources Investigation 78-113 Sediment Transportation by the White River into Mud Mountain Reservoir	June 1974-June 1976
Design Memorandum No. 20B—The Mud Mountain Dam Master Plans, Phase III	September 1974
Periodic Inspection and Continued Evaluation Report No. 5—Inspection of:	22 April 1975
Design Memorandum No. 1C—Mud Mountain Master Plan	April 1976
Design Memorandum No. 2—Visitors Center	May 1976
Design Memorandum No. 3—Water Treatment Plant	September 1976

Subject	Date
Real Estate—Design Memorandum No. 8—Revised Guide Taking Line	September 1976
Means of Improving the Capability and Quality of the Water System—Harstad Accounts, Inc.	March 1977
Periodic Inspection Report No. 6—Inspection of:	April 1977
Design Memorandum No. 4—Stabilizing Right Downstream Bank	September 1977
Design Memorandum No. 5—Construction of Road on Upstream Face of Dam	September 1977
Design Memorandum No. 6—Stop Log Hoist for 9-Foot Tunnel	September 1978
Design Memorandum No. 7—Rehabilitation of Apron Structure for 9-Foot Tunnel	August 1979
Supplement No. 1 to Design Memorandum 1C—Landscaping and Parking Improvements	October 1979
Design Memorandum No. 12—Supplement No. 9, Rockfill Dam, Spillway Outlet Works and Related Project Facilities	--
Operational and Maintenance Manual	June 1981
Emergency Preparedness Brief with Dam Break Flood Inundation Maps	April 1982
Periodic Inspection Report No. 7—Inspection of:	5 May 1982
Effect of Mud Mountain Dam Regulation on Sediment Movement in the White River, Northwest Hydraulic Consultants, Inc.	July 1983
Design Memorandum No. 25—Earthquake Analysis of Mud Mountain Dam	September 1983
Reconnaissance Report for Mud Mountain Dam, Dam Safety Assurance Program	April 1984

Subject	Date
Preliminary Reconnaissance Report on Seepage Studies at Mud Mountain Dam	20 August 1985
Reconnaissance Report on Seepage Control, Mud Mountain Dam	13 December 1985
Real Estate—Design Memorandum No. 27—Contractor Staging Area Dam Safety Assurance Program, Mud Mountain Dam	January 1986
General Design Memorandum No. 26—Dam Safety Assurance Program, Mud Mountain Dam	July 1986
Supplement No. 1 to General Design Memorandum No. 26—Mud Mountain Dam Core of Dam Seepage Control Measures	July 1986
Feature Design Memorandum No. 28—Outlet Works Modifications, Mud Mountain Dam	February 1989
Soletanche "As-Built" Final Report Concrete Cutoff Wall	1990

SECTION I

INTRODUCTION

SECTION I - INTRODUCTION

A. Location and Description of Project. Mud Mountain Dam is located on the White River, near approximately 40 miles southeast of Seattle, Washington, just inside the western Cascade Mountain front, at the eastern edge of the Puget Sound basin (Plate 1). The dam is used solely as a flood control facility for Puyallup, Sumner, Tacoma and the lower White and Puyallup river basin. The adjacent Puyallup and Carbon Rivers to the south are unregulated and converge with the White River in Sumner.

The dam is a zoned, 425 ft. high, earthfill structure consisting of a central dam core, flanking transition zones and sluiced rock shells. Elevation at the top of the dam (pre-contract) is 1250, with a flanking concrete chute spillway (crest El. 1215) on the right abutment (Plate 2). Normal riverflow is passed through two controlled 2,000 ft. long tunnels in the right bank. The dam embankment was constructed from 1939 to 1941 and was at that time the highest embankment dam in the world.

B. Construction and Study Authorizations. The construction of the dam a flood control dam at Mud Mountain was authorized by the Flood Control Act of 1936. The flood Control Act of 1938 provided for the operation and maintenance (O&M) of the completed project, which is under the supervision of the Seattle District, U.S.

(to pg I-2

Army Corps of Engineers. Construction and O&M of the dam and its recreational facilities was authorized by the Flood Control Act of 1944.

Study authority for the current project was in accordance with ER 1110-2-417, Project Operation Major Rehabilitation Program and Dam Safety Assurance Program, dated November 30, 1980. Construction of the seepage cutoff wall is pursuant to Supplement No. 1 of General Design Memorandum No. 26.

C. Purpose of Report. This foundation report is prepared and submitted in accordance with ER 1110-1-1801, dated December 15, 1981. It documents construction procedures and foundation conditions encountered on this unique project. The information contained herein will be useful for future work on the embankment, or for planning purposes on projects with similar design requirements.

D. Statement of Problem: In 1980 a single open piezometer tube (P-40) was installed in the core of the dam at its deepest point. The piezometric surface was monitored through 1984 with some unsettling results. Through the 4 year period, the piezometer responded progressively faster to fluctuations in seasonal pools. This was disturbing in light of the fact that design core permeability was on the order of $1X10^{-6}$ cm/sec. Seventeen additional piezometers were installed in 1985-86 to verify and characterize the problem. These borings encountered loose zones, heaving, "clean" sands and gravels and suspected "voids". Gradation tests on samples taken from these borings suggested they were core materials from which the fine sand sizes had been removed.

Subsequent information gained from the additional piezometers corroborated the suspicion that the fines were progressively being removed from the dam core by seasonal pool raises. It is suspected that water from each pool raise infiltrated a network of cracks and loose zones in the upstream face of the core, then pulled the finer faction upstream upon lowering.

E. Location of Structure. Most of the cutoff wall alignment is located 10 feet upstream and parallel to the dam axis as shown on Plate 2. The wall is 807.5 feet long, 32" to 40" wide and 20 to 402.6 feet deep, depending on bedrock elevation, including a minimum 15 feet embedment normal to bedrock surfaces.

F. Contractor and Contract Supervision. Construction of cutoff walls by the slurry trench method has been proven reliable and effective in several large Corps of Engineers and Bureau of Reclamation dam remediation projects, along with many similar private, domestic and foreign projects. Seattle District requested prequalification of bidders. Four companies bid on the (revised) contract in July 1988; Soletanche, Inc., Bachy/Bauer/Raymond/Green (Joint Venture), S.A. Healy Co. - I.C.O.S. SPA (Joint Venture), and Bencor Petrifond (Joint Venture).

The Government estimate for the work was $31,676,000.00 and was based on conventional clamshell excavation utilizing steel guide members. The contract was awarded July 26, 1988 to Soletanche, Inc. for $19,948,900.00 with

Notice to Proceed issued August 12, 1988. This low bid was based on Soletanche's anticipated use of their state-of-the-art excavator called the "hydrofraise".

On site personnel included the following:

Soletanche - Management	COE Resident Staff
Jacques Levallois - Project Manager	Larry D. Ems - Resident Engineer
Marc Van de Eynde - Q.C. Manager	Hiroshi Eto - Assistant R.E.
Etienne Dietsche - Proj. Superintendent	Matthew Satter - Resident Geologist
Brent Jones - Field Engineer	Ken Forbes - Concrete Specialist
Merrall Sims - Field Engineer	Dick Wilsey - Const. Representative
Jean-Luc Gobert - Q.C. Manager	Stu Wright - Const. Representative
Phil Fachan - Site Superintendent	Jacqueline Perreault - Secretary

SECTION II

GEOLOGY

SECTION II - GEOLOGY

A. Areal Geology. The project is located on the edge of the western Cascade Mountain front, where the White River Valley joins the Puget Sound Basin. The unique and complex topography and geology which influence the whole project area are a product of multiple Pleistocene glaciations in an ice border environment and periodic deposition of pyroclastic mudflows (lahars) and debris flows originating from the present and ancestral Mount Rainier volcanic center. This Pleistocene and Holocene geologic history has resulted in a complex of ice-marginal fill terraces and channels across and adjacent to the mouth of the valley and flanking areas of the Puget Sound Basin. It has further resulted in several diversions of the White River in this vicinity. At the dam site, the river channel lies in a sharp turn against the south valley side where the river has cut a deep canyon through the Pleistocene units and into the underlying mudflows and volcanic bedrock. Bedrock is comprised of Tertiary volcanics which are dominated by andesitic lavas, breccias and agglomerates, with subordinate amounts of tuff and local zones of hydrothermal alterations.

B. Seismic Setting. The Puget Sound area is considered a region of moderate seismicity, a product of crustal and subcrustal events linked to the subduction of oceanic crust beneath the Puget Sound basin. Historic macroseismicity and microseismicity have occurred on, or near the Buckley fault and historic

microseismicity has occurred on, or near both the Grass Mountain fault and the Mount Rainier lineament. The embankment has experienced shaking from both the 1949 and 1965 Puget Sound earthquakes with Modified Mercalli intensities of VII. Estimated site accelerations are on the order of 0.15 g (far field). There was no settlement of the embankment during these events although 1 to 1.5-in. longitudinal cracks opened up along the dam crest at the juncture of the core and rock fill (U.S. Army Corps of Engineers, 1949, 1965). The April 13, 1949 Richter magnitude 7.1 earthquake, epicenter in Olympia, opened a crack in the Mud Mountain Complex overburden materials near the intake structure to a depth of 10 feet. The seismicity of the region is thoroughly characterized in DM25 "Earthquake Analysis of Mud Mountain Dam" (COE, 1983).

In 1983 the embankment was analyzed using a permanent displacement (Newmark) analysis for a peak acceleration of 0.45 g and a duration of 14 sec (greater than the values expected from the proposed maximum credible earthquake). The analysis indicated a maximum of 7 to 13 in. of permanent displacement at about 60 percent embankment height, which was an acceptable order of magnitude (U.S. Army Corps of Engineers 1983).

C. Site Geology. Mud Mountain dam site is situated in a steep, narrow canyon (80 feet wide at riverbed) comprised of Miocene-age volcanics overlain by a series of (probable) Pleistocene-age lahars, debris flows, water-laid tuffs and related fluvial deposits. Deposits of Pleistocene glaciations from Mt. Rainier ice caps overlie the earlier volcanics and debris flows. The Recent

(5700 yrs) Osceola mud flow caps Mud Mountain and directly overlies the Pleistocene glacial deposits (Fig. II-1 and II-2).

D. Stratigraphy.

1. Bedrock. The andesite, andesite breccia, agglomerate, lithic tuff, and local sedimentary materials belong to the Enumclaw Formation. The andesite and andesite breccias are typically welded into a competent, though jointed rock mass. The agglomerates are typically more massive than the andesite and form the majority of the river canyon cliff sections. Locally, however, the agglomerate can be less competent than the overlying lahar deposits. The lithic tuff exists as thin beds with localized intrusions of sedimentary material. The bedrock contact with the overlying, lower unit of the Mud Mountain Complex tends to be irregular.

2. Mud Mountain Complex. The Mud Mountain Complex consists of a series of pyroclastic mudflows (lahars), debris flows, water-laid tuffs, and related fluvial deposits approximately 200 feet thick and overlie the bedrock surface. Although a general correlation of three major units and several minor units is seen, individual stratigraphic units in the sequence are difficult to trace laterally for more than a few hundred feet. The bulk of the lahar material is a hard, highly plastic, cobbly, gravelly silt and clay with minor amounts of sand, wood fragments, and pumice. Commonly the basal portion of the Mud Mountain Complex consists of a fluvial boulder gravel and a discontinuous

water-laid tuff bed sandwiched between the lowest lahar unit and the thickest lahar unit.

The uppermost part of the complex is locally characterized by a bouldery debris flow which exhibits fluvial channeling at its base. Where the debris flow is missing, 10 to 15 feet of deeply weathered residium characterizes the top of the sequence. Close examination indicates microvesiculation and charred wood fragments in some zones indicative of the still hot nature of the material at the time of deposition. These materials are believed to be at least as old as middle Pleistocene and the noted lack of clasts from the modern Mount Rainier volcano suggest a source from an earlier volcanic center. In the south canyon wall the Mud Mountain Complex exhibits a far more fluvial and bouldery character and extends about 30 to 40 feet higher in elevation than on the north bank suggesting post-Mud Mountain Complex erosion prior to the deposition of the overlying Hayden Creek drift.

3. Glacial Deposits. While not encountered in the cutoff wall construction, the upper glacial units are discussed here for a comprehensive stratigraphic familiarization:

a. Hayden Creek Drift. Deposits of the Hayden Creek glaciation overlie the Mud Mountain Complex in this portion of the White River Canyon. On the south bank the drift consists of about 25 ft. of very dense gravelly clay containing slightly oxidized gravel overlain by 30 ft. of varved to thick-

bedded clays, thin turbidites, and peat beds. These are overlain by more than 100 ft. of oxidized gravels which rise to the high terrace surface on the south bank. On the north bank the bulk of the Hayden Creek drift probably represents a Cascade precursor to the Salmon Springs glaciation of the Puget Trough. While there is some evidence for earlier glaciations in this region, the deposits are spotty and have not been recognized in this portion of the canyon and thus have no bearing on the engineering geology of the project.

b. Vashon Drift. In the right canyon wall the Vashon drift is represented by a single unit of glacial outwash sand, gravel, and boulders above a nominal elevation of 1,240/1,250 feet. The materials tend to be loose and are highly pervious with a zone of perennial springs exiting the valley walls at the top of the underlying till or lakebeds of the Hayden Creek drift. The Vashon drift was deposited between 14,000 and 12,000 years ago as ice moving southward from sources in Canada encroached on the Cascade Mountain front. While this ice did not occupy the present position of Mud Mountain, the associated ice marginal stream and lake deposits have major implications on the engineering geology of the Mud Mountain reservoir.

4. Osceola Mudflow. Capping virtually all of the flat-topped Mud Mountain is a 5,700 year old mudflow which varies in thickness from 2 to 30 ft. and consists of a heterogeneous mixture of boulders through clay (montmorillinite) material, together with occasional logs and smaller wood fragments. The Osceola mudflow can be traced upstream well above the mouth of the Clearwater

River. The mudflow provides a relatively impervious cap over the Vashon outwash and tends to pond water in low areas on the ridge top.

E. Structure. Major geologic structures identified in the area include the Grass Mountain, Buckley, Boise Creek-Clearwater River faults, and the Mt. Rainier lineament. In this area the Grass Mountain Fault may well be the southern boundary of the Olympic-Wallowa lineament, a regional northwest-southeast trending zone of transverse geologic structure which separates the North Cascades from the southern volcanic part of the range.

1. Bedrock. The bedrock is crossed by numerous local faults which may be related to the period of volcanic deposition. Major faults typically dip upstream and generally strike Northeast-Southwest (Figures II-3 through II-6 and Table II-1). The andesite and andesite breccias are characterized by closely to moderately spaced joints while jointing in the agglomerates is more likely to be more widely spaced. The thin beds of lithic tuff provide the major clue to structural attitudes.

2. Mud Mountain Complex. Shear zones and fissures occur locally in the Mud Mountain Complex materials. Some cracks are known to have been induced by recent seismic events and there is evidence of prehistoric displacements downstream of the spillway as evidenced by extensively weathered fissure boundaries on down-dropped blocks exposed in the canyon walls.

Stress relief cracks have been noted in and near the canyon walls. The

cracks are apparently confined to the lahar sequence of the Mud Mountain complex. In 1948 a 200 ft. long crack was discovered in the narrow overburden spur separating the south side of the spillway from the downstream canyon. In 1974 a 75 ft. high mudflow cornice on the canyon wall, 200 ft south of the earlier crack, developed cracks high on the slope and was removed. A 10 ft. deep crack opened up above the intake structure as a result of the 1949 earthquake (U.S. Army Corps of Engineers, 1949) and material outboard of the crack was removed. In 1984, while drilling high on the canyon wall above the intake structure, some 1,800 gal of drilling fluid were lost into the Mud Mountain complex in a zone of otherwise impervious lahar deposits. No fluid could be seen exiting on the steep slope below, and a stress relief feature parallel to the canyon wall appears a reasonable explanation. When drilling piezometer PZ-4 (a.k.a. Piezometer 110) during this contract, circulation was rapidly lost and never regained at 150 feet. Again, no trace of the drill fluid was evident in the canyon walls to the south.

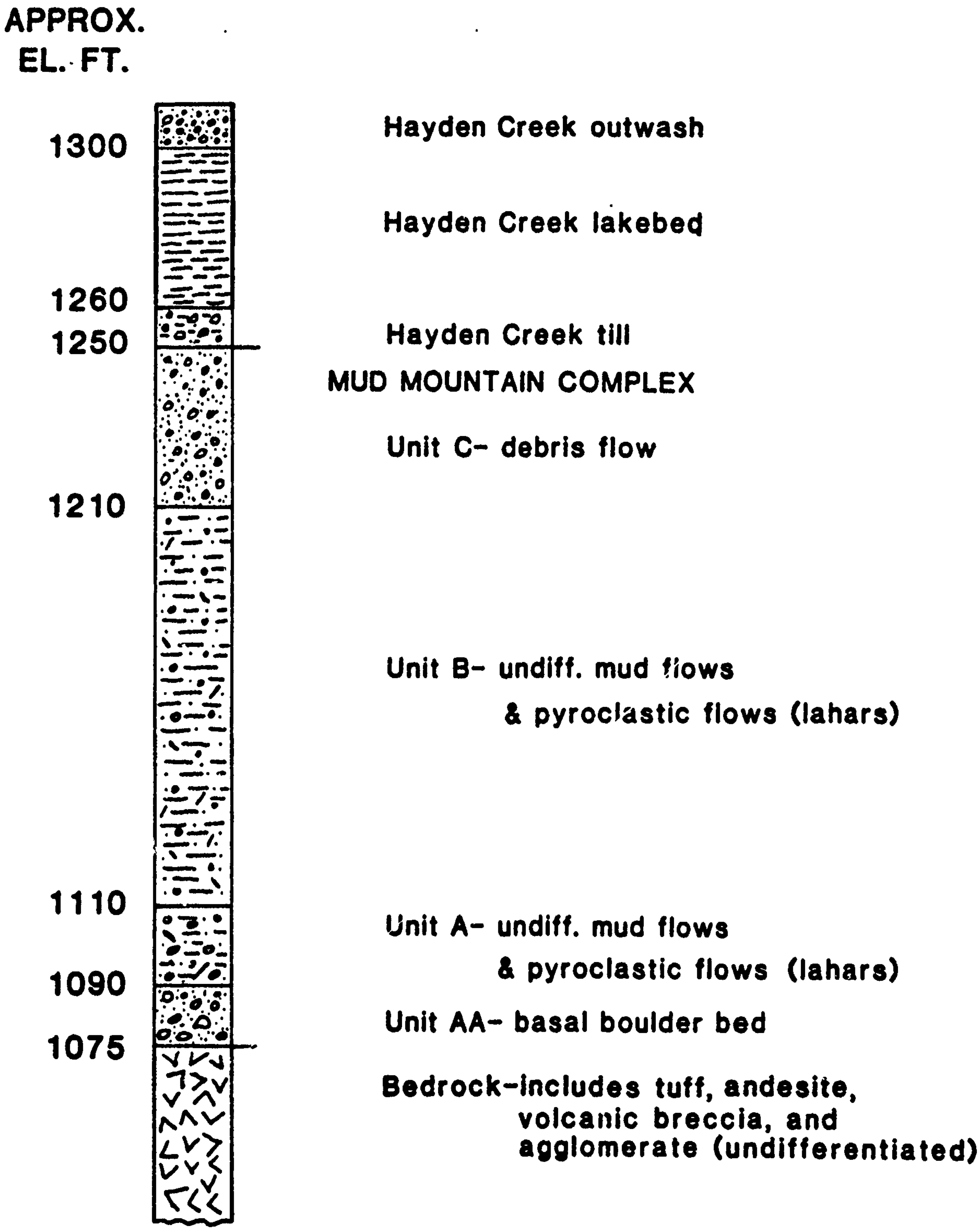
F. Weathering.

1. Bedrock. The andesite agglomerates are typically the bedrock factions showing signs of hydrothermal alteration. During the original dam construction excessively weathered bedrock surfaces were barred and scaled off, while pockets or zones of weathered and punky material around fractures and shear zones were removed by dental excavation and backfilled with concrete.

2. Mud Mountain Complex. These overburden materials contain shear zones of sand, silt and ash that have been altered to clay minerals, while the gravel, cobbles and boulders vary from fresh, to totally weathered (exfoliated).

G. Groundwater. Since original dam construction, groundwater flowing in and through bedrock fractures and seams in the canyon walls has been recognized. This was especially true in the left bank since it receives drainage from the higher topographic elevations to the south. Surface flows into the project site include Upper Cascade Creek, which drops as a falls into the normally drafted reservoir and Lower Cascade Creek, which enters the site on the top of the dam at the far left valley, where the left dam axis is keyed. Flows from Lower Cascade Creek are collected by a culvert, then diverted and dumped on the upstream face of the dam, where it dissipates into the rock shell. When there is no pool behind the dam, much of the ground water present in the left canyon wall is believed to be derived from the portion of these surface flows which percolate down into bedrock, then "daylights" in the left canyon wall at the core/bedrock boundary. During high pools, water is transmitted downstream through the network of joints and fractures in both canyon walls. These groundwater sources may have had significant erosional influence on the core along the rock contact, especially in the left bank area, which reflected faster piezometric responses to reservoir pools. Water trapped in the canyon walls adjacent to the core subsequent to a pool draft, could have accentuated the problem by flushing fines back upstream.

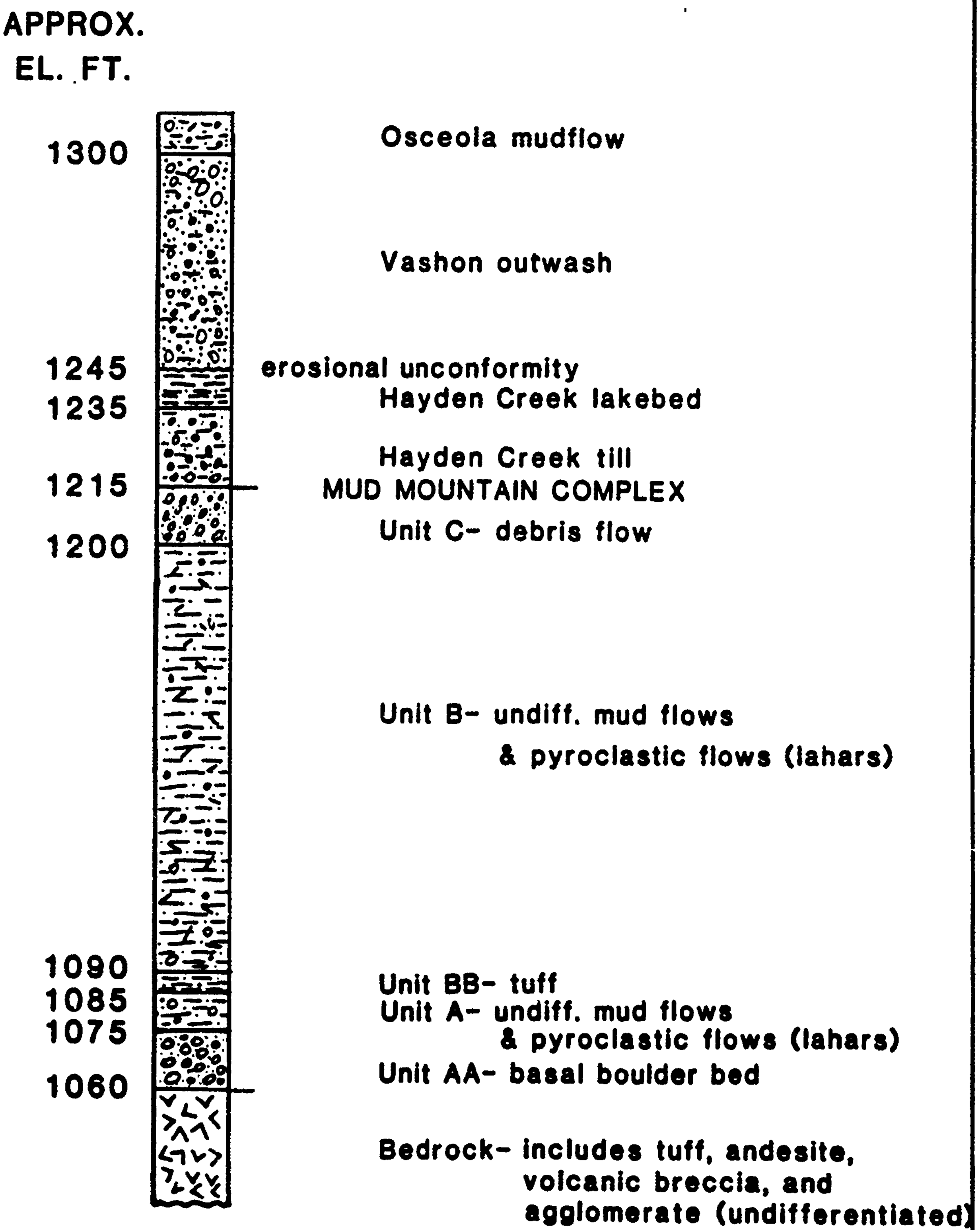
STRATIGRAPHIC COLUMN
LEFT ABUTMENT
UPPER CASCADE CREEK



Not to scale

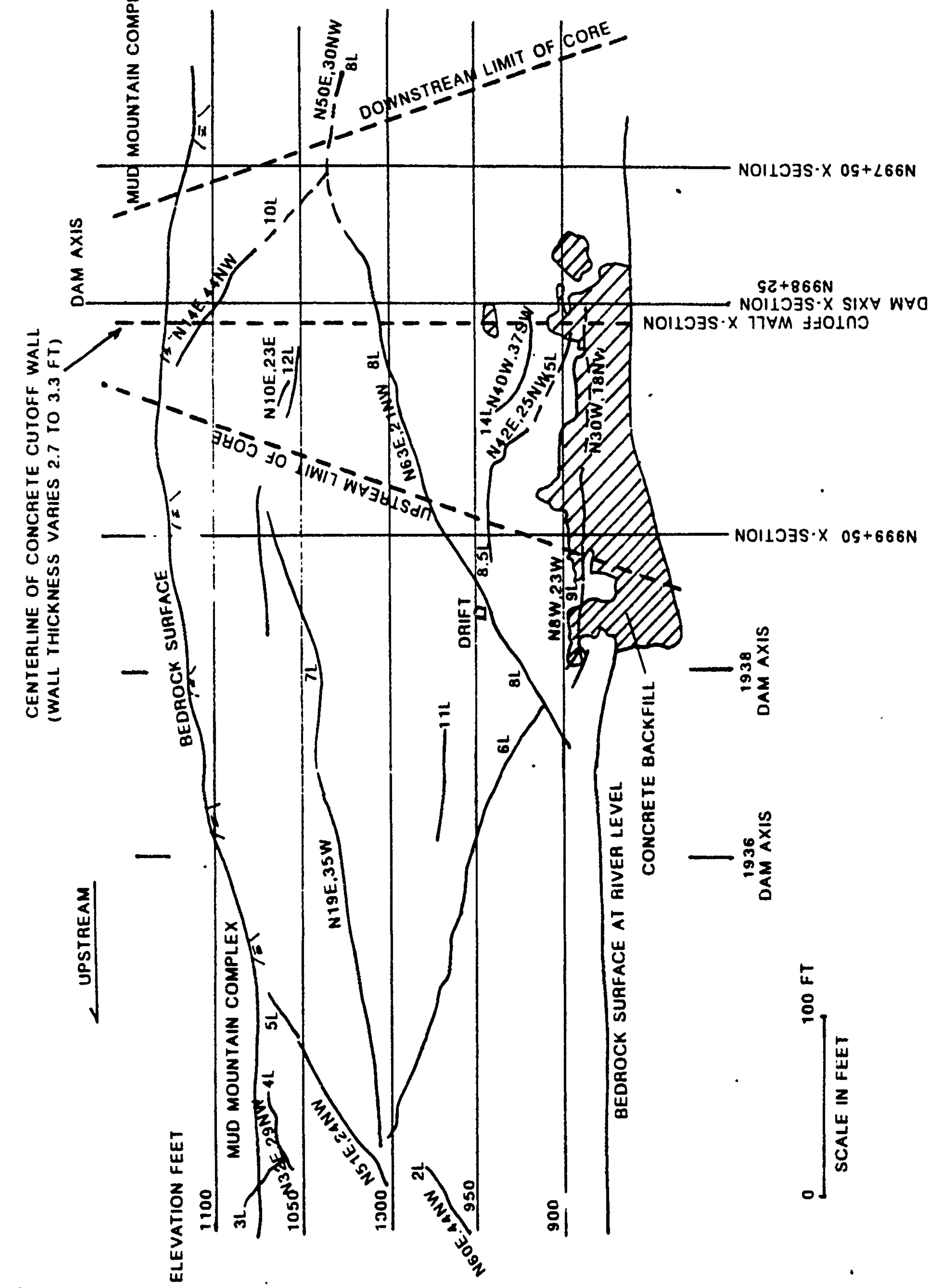
FIGURE II-1

STRATIGRAPHIC COLUMN
RIGHT ABUTMENT
UPSTREAM FROM SPILLWAY



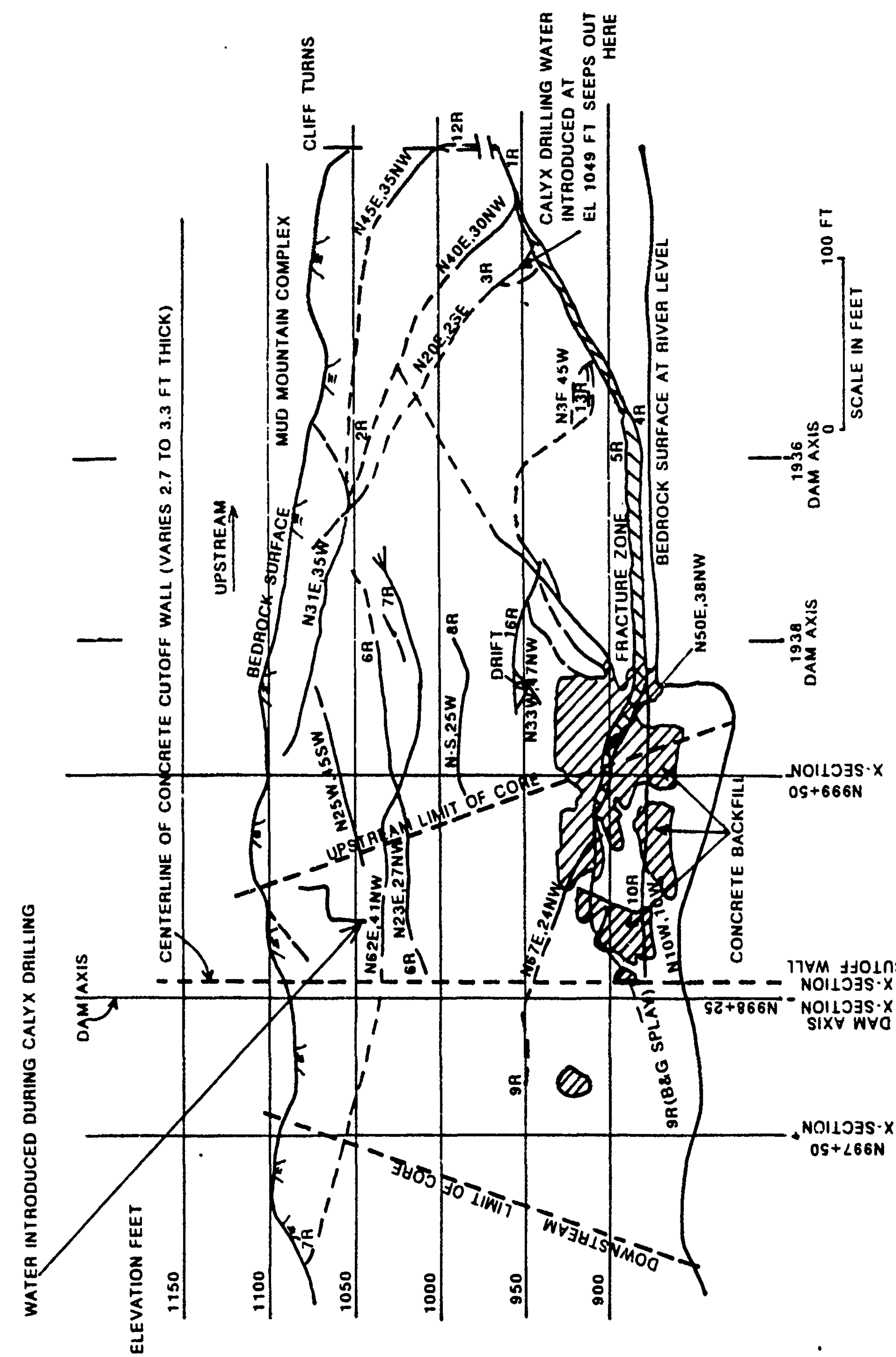
Not to scale

FIGURE II-2



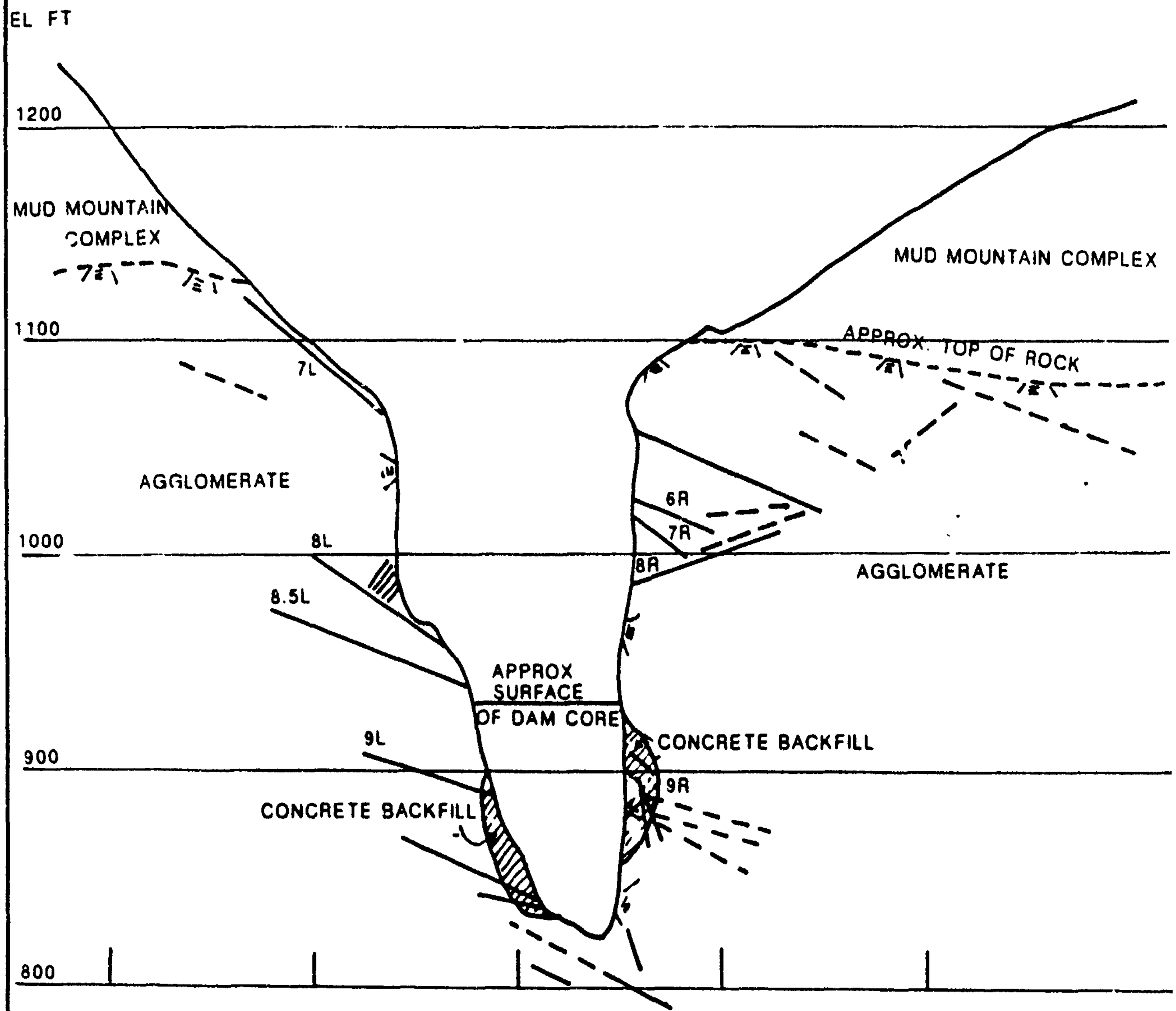
FAULTS ON LEFT CANYON WALL

FIGURE II-3



FAULTS ON RIGHT CANYON WALL

FIGURE II-4



GEOLOGIC CROSS SECTION

SECTION STA N999+50

VIEW DOWNSTREAM

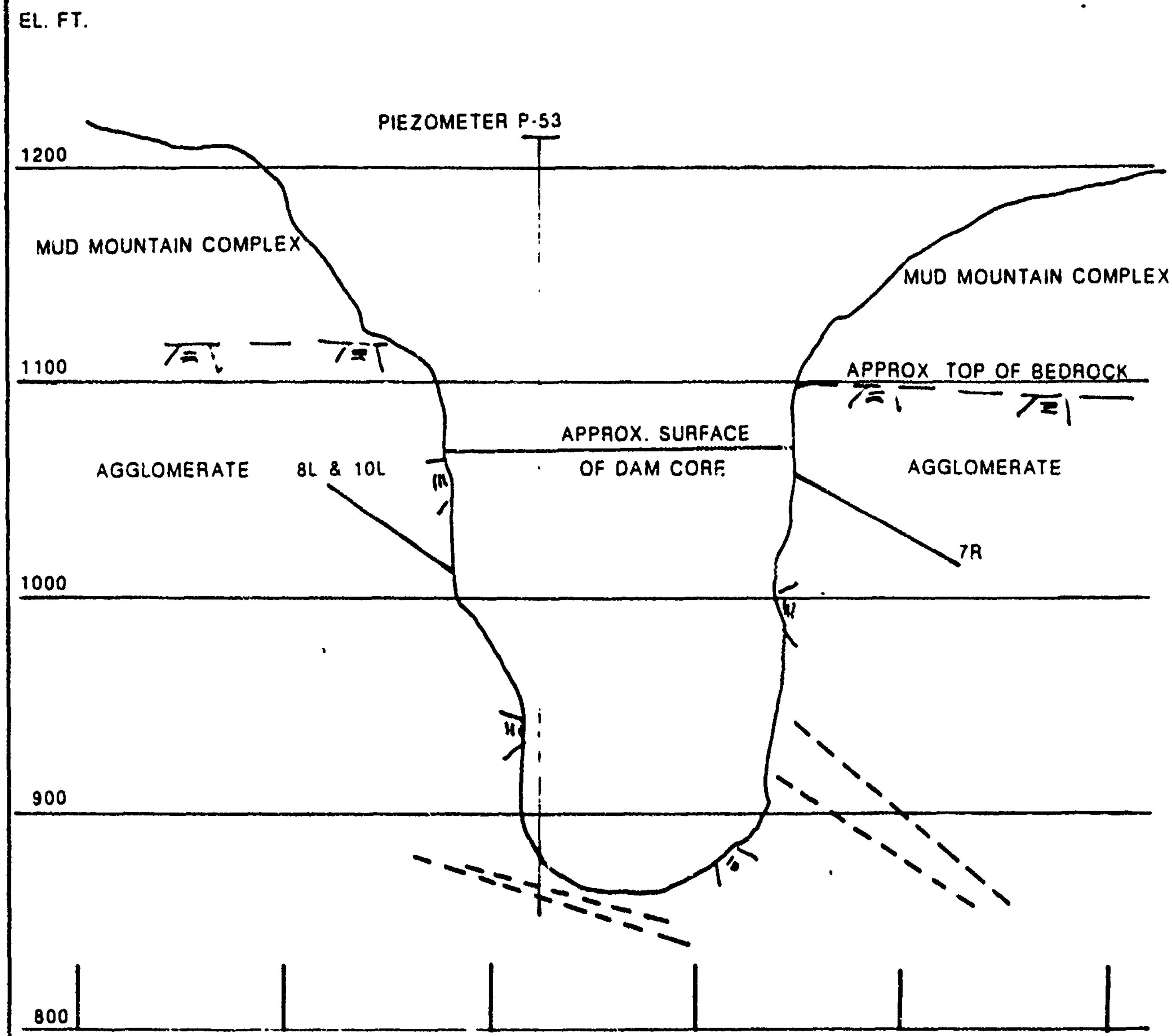
NOTES.

1 FAULT DIPS ARE APPARENT DIPS ON THIS CROSS SECTION PLANE (S54E)

2 CROSS SECTION IS PARALLEL AND 115 FEET UPSTREAM OF CUTOFF WALL

0 100 FT
SCALE IN FEET

FIGURE II-5



GEOLOGIC CROSS SECTION

SECTION VIEW STA. 997+50

VIEW DOWNSTREAM

NOTES

1 FAULT DIPS ARE APPARENT DIPS ON THIS CROSS SECTION PLANE (S54E).

2 CROSS SECTION IS PARALLEL TO AND 85 FT DOWNSTREAM OF CUTOFF WALL

0 100 FT

SCALE IN FEET

FIGURE II-6

Faults were recorded as primary, secondary and tertiary.

Name	Value	Description
Right Canyon Wall		
1R	Primary	At sta. N1002+60, el. 975', 5' blue gouge with slickensides, probably connects with 4R, 5R, 9R, and 10R.
2R	Tertiary	14" of crushed and oxidized gouge, slickensides
3R	Tertiary	1' to 8' crushed and oxidized gouge, no slicks.
4R,5R	Primary	Same as 1R description
6R	Secondary	1' of oxidized gouge
7R	Secondary	1' of oxidized gouge
8R	Tertiary	No gouge or slicks.
9R,10R	Primary	Same as 1R; occas. 4" wide zone crushed material; striations on all surfaces
12R	Tertiary	Tight to 1' of oxidized gouge, slicks run downdip
13R	Tertiary	Tight and no gouge
14R	Tertiary	Appears tight
15R	Tertiary	Tight and no gouge
Left Canyon Wall		
1L	Tertiary	Up to 0.7' oxidized gouge
2L	Tertiary	Up to 0.3' oxidized gouge
3L	Tertiary	Up to 0.5' oxidized gouge
4L	Tertiary	Up to 0.5' oxidized gouge
5L	Secondary	Tight, no water, slicks indicate a normal fault
6L	Primary	Tight, strikes approx N20W and dips 45NE
7L	Primary	Up to 0.2' open, slickensides, no gouge
8L	Primary	Blue clay and crushed oxidized gouge in zone 0.2' to 1.5' wide. In adit 0.2' to 0.8' oxidized gouge
9L	Primary	0.2' to 5' wide gouge and fractured crushed rock, variable
10L	Secondary	0.2' oxidized gouge

An exploration adit was driven along fault 8L on the left canyon wall. Within the adit, inclination of fault 8L was measured at 32 degrees and the fault walls were found to be highly polished. Fault width varied from 2 feet to 1/4 inch and filling was clay gouge and crushed rock. During the rainy season the fault segment in the adit remained dry for approximately two weeks, then ground water began percolating from it. While drilling a calyx hole on the right abutment drill water was lost at elevation 1049 feet. Two hours later the water emerged 300 feet upstream from a vertical crack, elevation 950 feet, associated with fault 3R.

TABLE II-1

SECTION III

EMBANKMENT/FOUNDATION EXPLORATIONS

SECTION III - EMBANKMENT/FOUNDATION EXPLORATIONS

A. Investigations.

1. Pre-1980 North Pacific Division, U.S. Army Corps of Engineers started initial investigations in 1936 and continued through dam construction completion in 1941. Bedrock and overburden conditions were investigated by diamond core drilling, calyx drilling, tunnels, test pits, and trenches.

2. Pre-Cutoff Wall (1980-87). Piezometers were installed during original dam construction but their location and any subsequently acquired data have been lost in antiquity. The first post-construction piezometer (P-40) was a single, open tube-type, installed in the deepest section of the dam embankment in 1980 (Plate 3). The piezometer responded fairly rapidly to pool fluctuations which suggested that the effective core permeability was higher than that anticipated by design ($1X10^{-6}$ cm/sec). This fact became even more disconcerting when in September 1984, the pool held a similar elevation (975 ft) for a comparable period of time as in 1982, and the piezometer showed a 6 ft. increase in piezometric surface (El.950 vs El.944). This suggested that not only was the present permeability higher than design, but that the effective permeability was increasing with time. The need for further definition of the problem led to the installation of an additional nine

piezometers in 1985, and eight more in 1986 (Plate 3). This drilling encountered a variety of soil conditions, all of which suggested the finer factions of the core being removed. The drilling encountered loose areas with possible voids as evidenced by the ease of advance. In one instance, the drill casing dropped 6 feet, of its own weight. Gradation tests on many of these materials indicate zones of loose, coarse sand and gravel present in the core.

3. Cutoff Wall Contract Investigations. Contract investigations were divided into embankment, overburden (Mud Mtn. Complex) and bedrock portions. The embankment portion was explored with an access shaft drilled 180 feet into the core, discussed at length in Section "V", and the piezometer drilling which occurred in the recompaction grouting zone in the vicinity of the cutoff wall. The original intent of the piezometer drilling and soil sampling was to identify "loose" zones in the embankment for monitoring. As a result of the recompaction of the core, the sampling portion of the installation was deleted from the contract. This deletion also lessened the impact costs resulting from the recompression grouting program.

The intent of the overburden and bedrock explorations was twofold: (1) to delineate the dam embankment/rock interface prior to hydrofraise excavation, especially in the critical steep canyon wall areas (two on the left bank, one on the right) and (2) determine the appropriate depth to which the cutoff wall should be carried, based on the quality of rock encountered at depth. Contractually, exploration of the bedrock was to take place within the 45 foot

periphery of the canyon profile (Plates 4 and 5), to identify zones of "punky" or highly fractured rock. It could then be determined to either remove it by means of hydrofraise panel excavation, or grout it up in a follow-on contract.

This exploration program was carried out concurrently with ongoing hydrofraise panel excavation, i.e., the hydrofraise would excavate to within 20 feet of the anticipated embankment/bedrock interface. It would then move off the panel while the core drill suspended a 4" ID casing full depth in the slurry-supported panel. The casing was "washed" through the remainder of the core material until it sat on bedrock, then an HX drill string was lowered inside to commence coring rock (Panels 13, 15, 17 and 19). The verticality of the suspended casing string could not be verified at greater depth, especially after it had washed through 20 feet of sediment. Therefore the accuracy of the embankment/bedrock contact was questionable. In practice, the hydrofraise ended up producing the most reliable bedrock profile, a product of it's "real-time" inclinometer and cutter head torque readout's ability to differentiate the easier embankment excavation from the harder rock. This information is presented on Plates 4 and 5 as a series of connected squares (overburden), or dots (bedrock). The data is shown for the primary or alternating one-bite panels only, as the readout from the hydrofraise torque data could not differentiate the embankment/rock boundary in closure panels with a background reading of the cutter wheels grinding out 4-6 inches of the adjacent primary panels.

Subsequently, the exploratory drilling and concrete quality control coring were carried out in a combined hole after panel concrete was placed. The panels were core drilled full depth and if bedrock exploration was desired, the hole continued to the target depth. Information on the exploratory and QC holes is given on Plates 5 and 6 and also Tables III-1 and III-2.

Core drilling was accomplished with a Longyear 44 truck-mounted drill, powered by a Detroit 353 diesel (120 hp) engine. It had wire-line retrieval capability and was fitted with a swivel ("H") head that would chuck 3.5 inch O.D. pipe. Drill circulation (and pressure testing concrete and rock) was handled with a diesel-powered Bean 70 pump. Core holes more than 200 ft. deep were drilled with an HX bit (3.65" hole), while those less than 200 ft. were drilled with an NX bit (2.98" hole). Drill depth capability was 1,600 feet.

Drilling a 400 ft. core hole within the confines of a 40-inch thick wall had problems. Early on, several holes left the panel into the core, so it became imperative to be able to control the direction of the drilling. The "Navidrill", as it is called, is manufactured by Eastman-Christensen Co., and utilized a down-the-hole motor, driven by water pressure at the end of non-rotating drill rods. This in turn, ran a face-discharge, diamond plug-bit through double-tilt U-joints. The Navidrill was powered by a trailer-mounted Bean-Royal 3-valve pump, with a GM 4-cylinder diesel motor. The Navidrill motor typically ran on 45 gpm at 300 psi. If a hole deviated too far, it would be grouted back to a point where correction could take place within the cutoff

wall panel, then directionally-drilled with the Navidrill. The amount of deviation was periodically monitored with an Eastman device, which was sent down the hole to photograph the orientation of the hole. Several time consuming corrections using backfill grout and directional-drill techniques led the drilling subcontractor (Boyles Bros., Salt Lake City) to increase the frequency of his hole monitoring. He detected and rectified problems early without having to grout back and re-drill. Quality of concrete is discussed in Section "XI".

As previously mentioned, it was important to delineate the core/canyon wall contacts prior to panel excavation, especially in the steep sections. Failure to do so could mean the 15 ft. embedment normal to the contact was not met. Prior to hydrofraise excavation, drill hole data from the gravity grouting program (discussed in Section IX) showed some discrepancies with respect to the contact in the lower left canyon wall. The bedrock contact in gravity grout holes 418 and 419 suggested a canyon bottom 15-20 ft. narrower (contact further to the right) than holes 415, 416A, 416B, and 417. An interpretation of these mixed results was aided by a review of several original dam construction photographs. It was found that the lower left canyon wall had a rather innocuous recess which just happened to fall on the cutoff wall alignment. The decision was made to deepen panel 127 from 357.5 ft. to 400 ft. to guarantee a minimum 15 ft. cutoff wall embedment. In the absence of a good photographic record for reference, it was possible that panel 127 would have been terminated at 357.5 ft., giving only 5-7 feet of cutoff wall embedment.

B. Engineering Characteristics of Overburden. As previously mentioned, the Mud Mountain complex consists of a series of pyroclastic mud flows (lahars), debris flows, water-laid tuffs and related fluvial deposits. The bulk of the mudflow material is a hard, highly plastic, cobbly, gravelly silt and clay (CH, MH, GC, and GM), with minor amounts of sand, wood fragments and pumice. Natural moisture content of these materials is 30 to 50 percent with the dry unit weights range from 60 to 90 pounds per cubic foot. Laboratory testing indicates preconsolidation of the material, which is consistent with the geologic record. The complex contains a large content of cobbles in the 6-9 inch plus range, as well as numerous boulders (to 6 foot diameter). Compressive strengths on the cobbles and boulders range from 680 to 27,200 psi.

C. Engineering Characteristics of Bedrock. Bedrock in the project area consists of andesite, andesite breccia, agglomerate and lithic tuff w/local intrusions of sedimentary material. Its competency and therefore its engineering quality is highly variable and unconfined compressive strengths can also range to over 20,000 psi. The andesites and andesite breccias are typically welded into a competent, though jointed, rock mass. The agglomerates form the majority of the cliff sections of the river canyon and are typically the weaker rock generally more massive but susceptible to hydrothermal alteration. Locally, it can actually be less competent than the overlying Mud Mountain Complex.

An incomplete tabulation of laboratory tests on samples of both the Mud

Mountain Complex and bedrock, taken prior to and during original dam construction, are available in "Analysis of Dam Design of Mud Mountain Dam" dated 6 May 1946. These tests included compressive strength, shearing strength and modulus of elasticity of selected bedrock samples.

EXPLORATORY HOLES

UPDATED 07 JUN 90

BORING NO.	DATES DRILLED	DEPTHS (FROM	TO)	TOTAL LF	REMARKS
89-DD-E15c-151	1-2 JUN 89	128.5	198.5	70	
89-DD-E17c-152	9-11 JUN 89	156.7	257.6	100.9	
89-DD-E13c-153	12-14 JUN 89	93	163	70	Drilled prior to panel place-
89-DD-E19c-154	14-15 JUN 89	172	283.3	111.3	ment (original intent).
89-DD-Q15C-155	19-21 JUN 89	146.5	151.5	5	
89-DD-Q17A-157	26-28 JUN 89	160.5	165.5	5	
89-DD-Q17C-158	28-30 JUN 89	174.8	180.0	5.2	
89-DD-QE15a-159	5-7 JUL 89	132.4	172	39.6	Tried to reclaim Schramm gravity
89-DD-E22--160	10-11 JUL 89	0	0	0	grout hole (pre-panel hole).
89-DD-QE39b-163	14 JUL 89	134.4	160	25.6	
89-DD-Q07B-164	18 JUL 89	71.8	76.8	5	
89-DD-Q08--165	19 JUL 89	77	82	5	
89-DD-QE35-168	25-26JUL&7AUG 89	170.6	250	79.4	
89-DD-Q41B-169	27-28 JUL 89	117.7	141.5	23.8	
89-DD-Q15A2-173	23-25 OCT 89	136.6	141.1	5	
90-DD-QE137-178	25JAN-20APR 90	385.5	450	64.5	Inclinometer casing installed
90-DD-QE119-180	2-9 FEB 90	186.7	280.3	93.6	
90-DD-QE126-182	8-16 MAR 90	324.5	429.9	105.4	Inclinometer casing installed
90-DD-QE123-183	19-31 MAR 90	278.7	399.9	121.2	Inclinometer casing installed
90-DD-QE130-187	3-12 APR 90	400.7	450.2	49.5	Inclinometer casing installed
90-DD-QE133-190	12-26 APR 90	393.5	425	31.5	Inclinometer casing installed
90-DD-QE139-196	26APR-9 MAY 90	377.5	424.5	47	Inclinometer casing installed
90-DD-QE143-198	1-4 MAY 90	193.6	400	206.4	
90-RD-133D-199	2 MAY 90	0	380		Inclinometer casing installed
	EXPLORATORY HOLES		----------	1269.9	

Boring No. = Year - Drill Type - Concrete Quality/Exploratory Hole, Panel No.,Bite - Hole No.

Example: 89-DD-QE15a-159 = 1989, Diamond Drill, Quality Concrete and Exploratory Hole, Panel No.15, Bite "a", Hole No. 159.

TABLE III-1

CONCRETE QUALITY BORINGS

UPDATED 07 JUN 90

BORING NO.	DATES DRILLED	DEPTH	
89-DD-Q15C-155	19-21 JUN 89	151.5	Extended to top of dam in wall.
89-DD-Q15B-156	23-26 JUN 89	146.7	
89-DD-Q17A-157	26-28 JUN 89	160.5	
89-DD-Q17C-158	28-30 JUN 89	180	Extended to top of dam in wall.
89-DD-Q15A-159	5-7 JUL 89	132.4	
89-DD-Q09B-161	12 JUL 89	82	
89-DD-Q43B-162	13-14 JUL 89	97.5	Extended to top of dam in wall.
89-DD-QE39B-163	15-17 JUL 89	134.4	Extended to top of dam in wall.
89-DD-Q07B-164	18 JUL 89	71.8	
89-DD-Q08--165	19 JUL 89	77	
89-DD-Q16--166	20 JUL 89	107	Out of panel 107 ft.
89-DD-Q16--167	21 JUL 89	159.2	
89-DD-QE35-168	25-26 JUL 89	170.6	Extended to top of dam in wall.
89-DD-Q41B-169	27-28 JUL 89	117.7	Extended to top of dam in wall.
89-DD-Q49B-170	28 JUL 89	83.6	Extended to top of dam in wall.
89-DD-Q12--171	7-8 AUG 89	100.8	
89-DD-Q11A-172	8-9 AUG 89	90.5	Extended to top of dam in wall.
89-DD-Q15A2-173	23-25 OCT 89	136.6	
89-DD-Q17B-175	30 OCT 89	12	
89-DD-Q07A-176	30-31 OCT 89	46.4	
89-DD-Q15C3-177	6-9 NOV 89	130.3	
90-DD-QE137-178	25JAN-20APR90	385.5	Inclinometer casing installed to 405 ft.*
90-DD-QE119-179	31JAN-2FEB90	107.7	Out of panel 107.7 ft.
90-DD-QE119-180	2-9 FEB 90	186.7	Extended to top of dam in wall.
90-DD-QE127-181	22FEB-8MAR90	237.9	
90-DD-QE126-182	8-16 MAR 90	324.5	Inclinometer casing installed to 332 ft. *
90-DD-QE123-183	19-31 MAR 90	278.7	Inclinometer casing installed to 349.5 ft.*
90-DD-QE130-187	3-12 APR 90	400.7	Inclinometer casing installed to 415 ft. *
90-DD-QE133-190	12-26 APR 90	393.5	Inclinometer casing installed to 403 ft. *
90-DD-QE139-196	26APR-9MAY90	377.5	Inclinometer casing installed to 395 ft. *
90-DD-QE143-198	1-4 MAY 90	193.6	Extended to top of dam in wall.
CONCRETE QC HOLES --------		TOTAL LF DRILLED 5274.8	

* Bottom in Exploratory Hole below QC hole.

TABLE III—2

SECTION IV

EXISTING DAM

SECTION IV - EXISTING DAM

A. Design. The dam is a zoned embankment, earthfill structure 425 ft. high. It consists of a central core of gravelly, sandy, silt and clay; flanking transition zones of 4" minus crushed diorite rock and; dumped and sluiced rock shells (Figure IV-1). An uncontrolled, concrete-lined chute spillway is situated adjacent to the embankment on the top of the right abutment. Water can be passed through two, 2,000 ft. tunnels in the right bank. The lower (El. 895 invert) 9 ft. diameter tunnel is controlled by means of a single radial gate at the entrance. The higher (intake El. 970) 23 ft. diameter tunnel passes river flows into three 8 1/2 ft. diameter penstocks toward the lower end of tunnel and are independently operated by Howell-Bunger valves at the discharge.

B. Construction History. The dam embankment was constructed from 1939 to 1941 and at the time was the highest embankment dam in the world. The proposed dam designs in the 1930's included a thin-arch concrete, concrete gravity, or concrete gravity thrust-type structures. These concepts were abandoned in favor of a rolled-fill earth dam. Subsequent to the commencement of construction, ongoing borrow-site investigations of the Osceola mud flow on top of Mud Mountain, revealed a small percentage of montmorillonite clay in the samples, which made it unsuitable for a rolled-fill application. At that point

the design was changed to a rock-fill structure with impervious core. The natural moisture content of the montmorillonite-rich flows was well above optimum, so the mudflow materials were blended with 80% sand and gravel and rotary kiln-dried prior to placement. There is a predominance of sand, rather than gravel above El. 1020, a result of differing borrow source locations. The moisture content of placed core lifts was insured by the erection of a large circus tent across the canyon during the winter months.

Restrictions on critically needed war materials deferred the installation of the 23 ft. penstocks, regulating valves, and valve house into a post-war contract. During this period, flows through the 23 ft. tunnel were regulated by a temporary orifice plug installed near the intake. Starting again in 1947, construction began for the completion of the 23 ft. tunnel appurtenances, fishway structures at Buckley, 9 ft. tunnel intake improvements and out buildings. Final project completion was June 1953.

C. Post-Construction Problems. Mud Mountain Dam was built using modern construction techniques, though state-of-the-art for embankment dam design has since changed. In this regard, the gradation of the existing transition zones does not meet currently accepted requirements for graded filters and, earthfill dam settlements and the phenomenon of soil arching is better understood.

1. Transition Zone. The purpose of the transition zones was to separate embankment materials of different gradation and permeability. It allowed

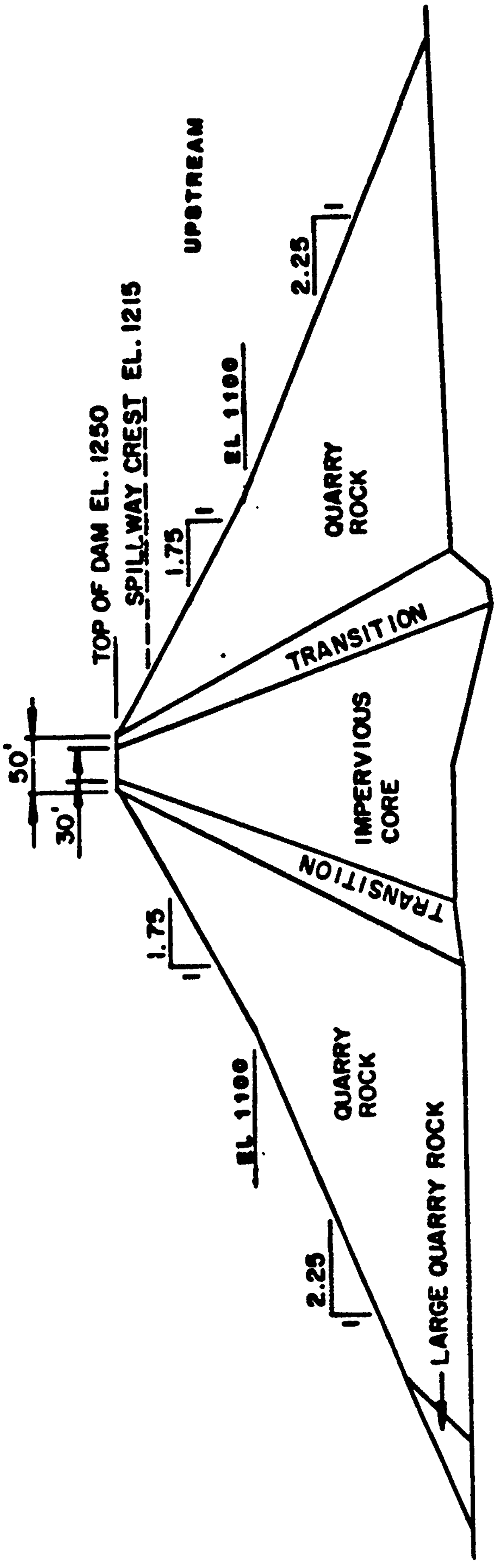
drainage while preventing finer material in the core from being washed away. The design intent was to provide two gradational zones of 4-inch minus crushed diorite rock, with the finer faction adjacent to the core. However, samples of material taken during construction were tested and the transition zone, as-built, did not end up with the specified zonation or gradation. 180 out of 299 control samples tested were coarser than specified (Figure IV-2).

2. Embankment Settlement. The second problem pertained to the complex nature of differential settlements occurring between the core, canyon walls and rockfill shells. Post-construction embankment settlement is normal and generally quantifiable. In zoned embankment dams, the compressibility of materials used for the core, transition zones and rock shells, along with the construction geometry, influence how much and where dam settlement occurs. In the case of Mud Mountain Dam, the rock shells and transition zones settled first, then became fairly rigid. Conversely, the core material (clay, silt, sand and gravel) being more compressible over the long term, continued to settle between the steep canyon walls and rock shells.

As the core continues to differentially settle, it can shear along boundarys parallel to it's confining medium. This manifests itself as cracking, either longitudinally along the axis of the dam, between the core/rock shell and/or transversely, parallel to the core/canyon interface above the steep "stair step" grade breaks. It was the anticipated presence of longitudinal and (especially) transverse cracking at depth, that was a pre-

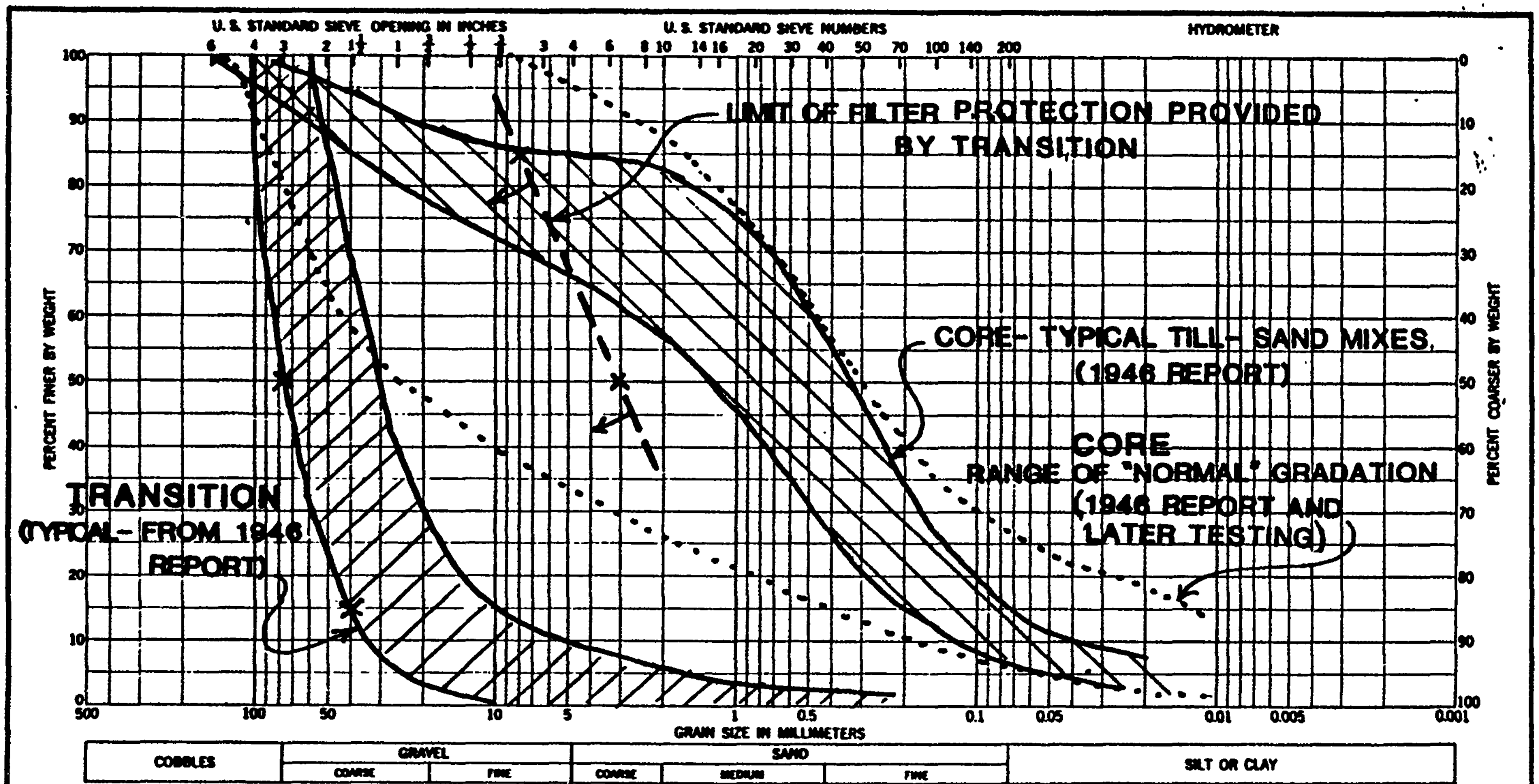
cutoff wall contract concern. In this scenario, if low stress zones were associated with this cracking and the soil pressures were less than hydrostatic, a cutoff wall panel filled with slurry that intersected a crack could hydrofracture to the rock shell, destabilize the trench and remove more core material, further weakening it. The purpose of the contract access shaft was to examine the core at depth, to determine if this type of cracking did, in fact, occur.

The differential settlement of the core within the confines of it's rigid boundarys can precipitate another phenomenon. In this scenario, as the core settles, it arches longitudinally between the confining canyon walls and/or transversely between the rock shells. Longitudinal arching would be more pronounced in the deeper portions of the core between the steep canyon walls. If this arching does occur, further settlement of the core above the arch is inhibited, while settlemnt below may continue, due to saturation from pool fluctuations. Hence, a low stress zone develops below the arch and soil pressures become reduced to the point where induced hydrostatic pressures from impounded pools might exceed soil pressures in these sub-horizontal planes. During this time these zones are vulnerable to saturation and hydrofracture. Additionally, fines can be progressively removed from hydrofracture cracks or low stress zones and sucked back upstream upon pool drafting. These planes of low stress or cracking could also transmit full reservoir pressures to the downstream side of the core, accelerating the loss of fines, producing even larger flow paths. The creation of low stress zones and suspected flushing of core fines through the transition zone from repeated pool raises probably worked interdependently.



EMBANKMENT SECTION

FIGURE IV-1



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

GRADATION CURVES

Project MUD MOUNTAIN DAM
CORE & TRANSITION
MATERIALS

Date Oct 85

ENG FORM 1 MAY 63 2087

FIGURE IV-2

SECTION V

ACCESS SHAFT

SECTION V - ACCESS SHAFT

A temporary access shaft was constructed approximately 5 feet upstream of the cutoff wall alinement at station 14+00 to investigate the condition of the dam core. The access shaft was constructed to a depth of 180 feet below the top of the lowered dam (elevation 1240 feet N.G.V.D.) using a crane suspended 4-foot diameter auger with a reaming attachment which allowed drilling a 6-foot diameter hole. A truck-mounted auger was used for the first 97 feet of excavation. The hole was cased by hand labor in two reaches, each 2-foot long, using segmented steel liner plates. The plates were bolted together six to a ring, 2 feet long, by two workers in a suspended man basket. All work, including Government investigations, was done from the man basket and in some instances from a boatswain's chair. The shaft casing was designed to be suspended from a concrete collar placed at the top of the shaft. The first three rings of liner plate (9-foot reach) was secured to the concrete collar by 24 one-half inch studs. The support capacity of the collar was conservatively computed to be three times higher than the total dead weight of the shaft liner. The shear capacity of the studs was much greater than the bearing capacity of the collar.

Construction of the access shaft began on December 12, 1988 with excavation using a backhoe for placement of the concrete collar and embedment of the first 9-foot reach of liner plate. The concrete collar was placed on December 14, 1988 and drilling commenced on December 15, 1988 using a Hughes LLDH 15140

truck-mounted auger. The truck-mounted auger was used to a depth of 97 feet. An auger mounted on a Manitowoc 4000 crane with a specially fabricated 200-foot kelly bar was used to advance the shaft below 97 feet starting on 13 Jan 1989. The shaft was advanced to a depth of 145 feet before interference from piezometer P50 required contract modifications to be issued for removal of interfering portions of the 6-inch steel piezometer pipe. The first modification amounted to $2,080.00 for removal of pipe between elevation 1094 and 1083 (146 to 157 feet in depth) and the second amounted to $8,316.00 for removal of pipe between elevation 1083 and 1059 (157 to 181 feet in depth). Excavation of the pilot hole and removal of the pipe before reaming continued to a depth of 180 feet when the piezometer pipe interference would not allow further drilling of the 4-foot diameter pilot hole.

The investigations did not reveal any major flaws in the dam core. Voids found around the piezometer casing were attributable to over-drilling and not to pre-existing voids within the core. There were three areas which indicated some cracking and surface water infiltration. The cracking is probably due to differential settlement between the core and rockfill zones and within the core itself. Cracks attributable to such settlement were also discovered when the top of dam was lowered by 10 ft. to widen the work area. The crack surfaces were very well defined, oxidized by water percolating down from the surface. The presence of water is probably due to surface water migration because the water filled cracks occur higher in the core than normal pool levels. A test pool raise in 1974 held the pool at or above elevation 1,140 feet N.G.V.D. for

about 45 days and reached elevation 1,150 feet N.G.V.D..

Sandcone density tests were conducted in accordance with ASTM Standard 1556 from a depth of 58 ft. to 180 ft. ~~down~~. The average wet density was 129.8 pcf, the average dry density was 115.0 pcf, and the average moisture content was 13.2 percent (see table V-1). Bulk samples were taken about every four feet in depth and used for compaction tests on the minus 3/4 inch fraction using ASTM D698 method D. The compaction tests indicated the average maximum dry density was 119.1 pcf with a range of 117.7 pcf to 123.2 pcf. The optimum water contents for these densities ranged from 11.8 to 14.1 percent and averaged 12.9 percent. Since the sandcone tests were conducted where the soil contained very little gravel size particles, the density of the core material in these areas was about 95 percent of maximum. Relative density (max-min) tests were also conducted on the bulk samples and indicated an average maximum density of 105.4 pcf (93.7 to 110.7 pcf range) and an average minimum density of 83.2 pcf (73.0 to 89.2 pcf range). The max-min test results are tabulated in table V-2. Small cellophane bag samples from each quadrant of the excavation were taken every 4 ft. for moisture content, Atterbergs and classification. The results of these tests are shown in tables V-3 to V-8. Detailed test results are included in the appendix to this report along with daily inspection logs.

MUD MOUNTAIN DAM ACCESS SHAFT SANDCONE TESTS

Sample #	Date	Station	Elevation (ft)	Depth (ft)	Offset	lb/ft3,wet	lb/ft3,dry	Water content,%
0047-S-SC1	04-Jan-89	P-50	1132.0	58.0	North Wall	123.3	110.1	11.9
0047-S-SC2	05-Jan-89	P-50	1176.0	64.0	South Wall	117.5	102.5	14.6
0047-S-SC3	10-Jan-89	P-50	1164.0	76.0	North Wall	139.8	128.4	8.8
0047-S-SC4	11-Jan-89	P-50	1156.0	84.0	East Wall	122.8	109.0	12.5
0047-S-SC5	17-Jan-89	P-50	1137.0	103.0	East Wall	135.7	116.8	16.0
0047-S-SC6	20-Jan-89	P-50	1129.0	111.0	North Face	135.6	119.4	13.5
0047-S-SC7	20-Jan-89	P-50	1129.0	111.0	North Wall	136.9	121.8	12.3
0047-S-SC8	20-Jan-89	P-50	1129.0	111.0	South Wall	131.8	115.9	14.2
0047-S-SC9	25-Jan-89	P-50	1117.0	127.0	East Wall	116.3	101.3	14.4
0047-S-SC10	30-Jan-89	P-50	1109.0	141.0	North Wall	126.7	110.4	16.4
0047-S-SC11	23-Feb-89	P-50	1065.0	175.0	South Wall	124.6	111.5	13.9
0047-S-SC12	23-Feb-89	P-50	1065.0	175.0	East Wall	121.5	113.2	9.1
0047-S-SC13	28-Feb-89	P-50	1062.0	187.0	South Wall	146.2	129.3	13.0
0047-S-SC14	08-Mar-89	P-50	1105.0	135.0	East Wall	128.5	112.8	13.9
0047-S-SC15	10-Mar-89	P-50	1146.0	94.0	North Wall	133.7	117.0	13.6
						129.8 AVG	115.0 AVG	13.2 AVG

TABLE V-1

CENPD-EN-G-L (89-SC-761)

MUD MOUNTAIN DAM - ACCESS SHAFT

Relative Density Test on Bulk Samples

CENPS Bulk Sample No.	Elevation, ft.	Depth, ft.	Dry Density, pcf Minimum	Dry Density, pcf Maximum
1	1225	15	83.9	106.1
2	1219	21	88.2	109.9
3	1210	30	82.9	105.6
4	1195	45	79.0	98.3
5	1180	60	89.2	110.1
6	1165	75	85.1	107.2
7	1156	84	87.2	108.7
8	1141	99	77.8	101.1
9	1125	115	86.9	109.9
10	1137	103	73.0	93.7
11	1129	111	80.4	101.9
12	1129	111	83.1	105.6
13	1129	111	78.2	102.6
14	1098	142	84.5	107.2
15	1090	150	87.0	110.7
16	1065	175	84.6	107.2

Reference:
- ASTM D 4253-83, "Maximum Index Density of Soils Using a Vibratory Table."
- ASTM D 4254-83, "Minimum Index Density of Soils and Calculation of Relative Density."

TABLE V-2

Received : 9 Feb and 8 Mar 89

MUD MOUNTAIN DAM - ACCESS SHAFT

Water Content and Atterberg Limit Tests on Baggie Samples

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1065	North	14.5	--	--	--	--
	South	13.7	--	--	--	--
	East	12.0	--	--	--	--
	West	12.8	--	--	--	--
	Composite	--	NP	NP	NP	ML
1069	North	15.0	--	--	--	--
	South	15.3	--	--	--	--
	East	17.6	--	--	--	--
	West	14.5	--	--	--	--
	Composite	--	NP	NP	NP	ML
1073	North	13.2	--	--	--	--
	South	13.2	--	--	--	--
	East	12.8	--	--	--	--
	West	14.7	--	--	--	--
	Composite	--	27	17	9	CL
1077	North	14.8	--	--	--	--
	South	14.3	--	--	--	--
	East	14.3	--	--	--	--
	West	12.6	--	--	--	--
	Composite	--	26	16	10	CL
1081	North	12.8	--	--	--	--
	South	12.6	--	--	--	--
	East	11.1	--	--	--	--
	West	13.7	--	--	--	--
	Composite	--	NP	NP	NP	ML
1085	North	16.7	--	--	--	--
	South	12.1	--	--	--	--
	East	10.8	--	--	--	--
	West	10.0	--	--	--	--
	Composite	--	29	16	13	CL
1088	North	15.5	--	--	--	--
	South	14.2	--	--	--	--
	East	13.9	--	--	--	--
	West	13.6	--	--	--	--
	Composite	--	NP	NP	NP	ML
1093	North	14.7	--	--	--	--
	South	14.7	--	--	--	--
	East	15.4	--	--	--	--
	West	13.6	--	--	--	--
	Composite	--	34	20	14	CL

TABLE V-3

CENPD-EN-G-L (89-S-761)

Subject : Mud Mountain Dam - Water Content and Atterberg Limit Test Results

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1097	North	14.3	--	--	--	--
	South	14.3	--	--	--	--
	East	12.5	--	--	--	--
	West	15.2	--	--	--	--
	Composite	--	32	21	11	CL
1101 (2-1-89) date	North	17.8	--	--	--	--
	South	14.6	--	--	--	--
	East	16.9	--	--	--	--
	West	10.7	--	--	--	--
	Composite	--	29	19	10	CL
1101 (no date)	North	17.8	--	--	--	--
	South	17.1	--	--	--	--
	East	14.5	--	--	--	--
	West	18.3	--	--	--	--
	Composite	--	27	18	9	CL
1105	North	16.1	--	--	--	--
	South	15.8	--	--	--	--
	East	15.7	--	--	--	--
	West	16.7	--	--	--	--
	Composite	--	NP	NP	NP	ML
1109	North	15.5	--	--	--	--
	South	16.1	--	--	--	--
	East	16.6	--	--	--	--
	West	18.0	--	--	--	--
	Composite	--	25	19	6	CL-ML
1113	North	15.1	--	--	--	--
	South	13.2	--	--	--	--
	East	15.6	--	--	--	--
	West	15.1	--	--	--	--
	Composite	--	NP	NP	NP	ML
1117	North	14.3	--	--	--	--
	South	15.8	--	--	--	--
	East	14.6	--	--	--	--
	West	14.4	--	--	--	--
	Composite	--	NP	NP	NP	ML
1122	North	11.9	--	--	--	--
	South	13.3	--	--	--	--
	East	14.6	--	--	--	--
	West	13.3	--	--	--	--
	Composite	--	NP	NP	NP	ML

TABLE V-4.

CENPD-EN-G-L (89-S-761)

Subject : Mud Mountain Dam - Water Content and Atterberg Limit Test Results

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1125	North	14.0	--	--	--	--
	South	14.5	--	--	--	--
	East	14.7	--	--	--	--
	West	14.3	--	--	--	--
	Composite	--	NP	NP	NP	ML
1129	North	14.2	--	--	--	--
	South	14.9	--	--	--	--
	East	14.9	--	--	--	--
	West	20.6	--	--	--	--
	Composite	--	NP	NP	NP	ML
1132	North	14.3	--	--	--	--
	South	15.7	--	--	--	--
	East	13.8	--	--	--	--
	West	15.0	--	--	--	--
	Composite	--	NP	NP	NP	ML
1137	North	18.7	--	--	--	--
	South	15.4	--	--	--	--
	East	15.5	--	--	--	--
	West	15.8	--	--	--	--
	Composite	--	28	19	9	CL
1141	North	14.3	--	--	--	--
	South	15.3	--	--	--	--
	East	16.6	--	--	--	--
	West	15.0	--	--	--	--
	Composite	--	NP	NP	NP	ML
1145	North	13.8	--	--	--	--
	South	13.7	--	--	--	--
	East	14.1	--	--	--	--
	West	14.3	--	--	--	--
	Composite	--	NP	NP	NP	ML
1149	North	15.6	--	--	--	--
	South	15.2	--	--	--	--
	East	15.5	--	--	--	--
	West	10.5	--	--	--	--
	Composite	--	NP	NP	NP	ML
1153	North	11.6	--	--	--	--
	South	12.7	--	--	--	--
	East	12.6	--	--	--	--
	West	10.5	--	--	--	--
	Composite	--	NP	NP	NP	ML

TABLE V-5

Subject : Mud Mountain Dam - Water Content and Atterberg Limit Test Results

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1156	North	12.6	--	--	--	--
	South	13.8	--	--	--	--
	East	13.7	--	--	--	--
	West	13.9	--	--	--	--
	Composite	--	NP	NP	NP	ML
1168	North	14.7	--	--	--	--
	South	15.8	--	--	--	--
	East	13.7	--	--	--	--
	West	14.7	--	--	--	--
	Composite	--	NP	NP	NP	ML
1169 3/	North	13.2	--	--	--	--
	South	13.7	--	--	--	--
	East	13.4	--	--	--	--
	West	14.6	--	--	--	--
	Composite	--	NP	NP	NP	ML
1173	4/	14.1	--	--	--	--
	4/	12.1	--	--	--	--
	4/	13.2	--	--	--	--
	4/	12.7	--	--	--	--
	Composite	--	NP	NP	NP	ML
1177	North	7.8	--	--	--	--
	South	13.3	--	--	--	--
	East	13.2	--	--	--	--
	West	14.1	--	--	--	--
	Composite	--	NP	NP	NP	ML
1180	North	15.0	--	--	--	--
	South	13.3	--	--	--	--
	East	14.3	--	--	--	--
	West	14.1	--	--	--	--
	Composite	--	27	17	10	CL
1185	North	14.9	--	--	--	--
	South	15.6	--	--	--	--
	East	14.8	--	--	--	--
	West	16.5	--	--	--	--
	Composite	--	23	19	4	CL-ML
1187	North	15.9	--	--	--	--
	South	15.5	--	--	--	--
	East	15.6	--	--	--	--
	West	15.7	--	--	--	--
	Composite	--	NP	NP	NP	ML

CENPD-EN-G-L (89-S-761)

Subject : Mud Mountain Dam - Water Content and Atterberg Limit Test Results

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1192	North	21.5	--	--	--	--
	South	24.7	--	--	--	--
	East	23.4	--	--	--	--
	West	16.2	--	--	--	--
	Composite	--	27	17	10	CL
1194	North	13.6	--	--	--	--
	South	13.8	--	--	--	--
	East	14.2	--	--	--	--
	West	15.6	--	--	--	--
	Composite	--	NP	NP	NP	ML
1202	North	14.3	--	--	--	--
	South	15.3	--	--	--	--
	East	15.2	--	--	--	--
	West	15.6	--	--	--	--
	Composite	--	NP	NP	NP	ML
1203	North	15.0	--	--	--	--
	South	14.8	--	--	--	--
	East	14.7	--	--	--	--
	West	18.2	--	--	--	--
	Composite	--	NP	NP	NP	ML
1206	North	14.7	--	--	--	--
	South	17.8	--	--	--	--
	East	14.6	--	--	--	--
	West	15.1	--	--	--	--
	Composite	--	NP	NP	NP	ML
1210	North	16.1	--	--	--	--
	South	14.1	--	--	--	--
	East	14.7	--	--	--	--
	West	19.7	--	--	--	--
	Composite	--	NP	NP	NP	ML
1212	North	13.2	--	--	--	--
	South	13.9	--	--	--	--
	East	14.1	--	--	--	--
	West	14.0	--	--	--	--
	Composite	--	NP	NP	NP	ML
1219	North	15.8	--	--	--	--
	South	15.3	--	--	--	--
	East	15.9	--	--	--	--
	West	15.6	--	--	--	--
	Composite	--	28	17	11	CL

TABLE V-7

CENPD-EN-G-L (89-S-761)

Subject : Mud Mountain Dam - Water Content and Atterberg Limit Test Results

Sample Elevation	Sample Location 1/	Water Content, %	Atterberg Limits, % Liquid	Plastic	Plastic Index	Unified Soil Classification 2/
1225	4/	14.7	--	--	--	--
	4/	15.2	--	--	--	--
	Composite	--	NP	NP	NP	ML
-- 5/	North	13.6	--	--	--	--
	South	13.8	--	--	--	--
	East	14.2	--	--	--	--
	West	15.6	--	--	--	--
	Composite	--	26	16	10	CL
1176-1175	1 of 2	13.9	NP	NP	NP	ML
(Jar)	2 of 2	14.3	NP	NP	NP	ML
1140	Jar	16.0	NP	NP	NP	ML
1139.5	Jar	15.4	NP	NP	NP	ML

Notes:
1/ Composite samples were composed of combined North, South, East and West samples.
2/ Atterberg Limit tests are performed on the minus No.40 fraction of the sample. The classification is for that portion only, not the entire sample.
3/ This sample was not shown on NPD Form 300, Sample Transmittal.
4/ No sample location shown on bag.
5/ No sample elevation shown on bags.

Received: 9 Feb and 3 Mar 89

TABLE V-8

SECTION VI

CONSTRUCTION

SECTION VI - CONSTRUCTION

A. General Overview. The main feature of the contract was the construction of a Seepage Control Cutoff Wall from elevation 1253 down to and embedded 15 ft. into bedrock. The Seepage Control Cutoff Wall contract also included raising the dam from elevation 1250 to 1257, extension of existing roads on the upstream and downstream face of the dam to the new top, construction of earth retension structures along the top of dam road extensions (Hilfiker Wall, Eureka, CA) realignment of the spillway access road, and constructing a temporary steel cased exploratory access shaft into the core of the dam. Incidental features included new piezometer installations, extension of selected existing piezometers, new inclinometer installations, investigative coring of the foundation bedrock and quality control coring of the concrete cutoff wall.

B. Site Preparation. The cutoff wall construction required large waste areas for disposal of material excavated from the core of the dam and slurry that could not be treated for reuse. The northern waste area was used for untreatable slurry disposal while the southern waste area was used for excavated materials. Realignment of the spillway access road required some 99,000 cy of excavation. The excavated material was disposed of in the waste areas. The size and number of heavy equipment anticipated to be used on top of the dam at the height of construction required that the top of the dam be lowered to elevation 1240 (10 ft. below existing) to provide a wider work

platform. The non-rock materials removed during the dam lowering (about 30,000 cy) was also to be disposed of in the waste areas. Two areas were clear cut near the Resident Office to provide over 10 acres of land for waste disposal (see plate 2). Another 4.5 acres was clear cut just west of the waste areas to provide a staging area which the contractor used to construct slurry holding ponds (four ponds each capable of storing 630 cy each), a slurry mixing station, concrete aggregate stockpiles, on-site concrete batch plant, and miscellaneous equipment storage.

Some of the material removed off the top of the dam was used to build an access ramp up off the spillway floor to the top of the work platform. The ramp also allowed the cutoff wall excavation equipment to work from a level platform straight off the top of the dam to excavate and place 90 ft. of the cutoff wall that extended out under the spillway.

C. Bentonite Slurry. Soletanche used 100 percent Wyoming bentonite (National Premium 90 bentonite) supplied by Baroid Minerals and Chemicals. Five primary characteristics were measured for slurry quality control: density, flowability, caking, chemical environment, and sand content.

The density of the slurry determines the hydrostatic pressure exerted on the trench walls. The hydrostatic pressure is the main stabilizing factor keeping the trench walls from caving. Once introduced into the trench, the density changes as native soils mix in and become suspended in the slurry (the

density increases). If the density becomes too high, the placement of tremie concrete will be impeded. The tremie concrete is not able to flow as readily and may result in open honeycombs and voids (same result as if a low slump concrete was used). The contract specification required a minimum bentonite concentration of 6 percent (minimum density of 64.3 pounds per cubic foot) by weight of water. During excavation the slurry density was allowed to go up to 90 pcf. Prior to concrete placement the slurry required a sand content less then 1 percent and slurry density less than 85 pcf.

The flowability is measured by four tests: apparent viscosity, plastic viscosity, Marsh funnel, and gel strength. High viscosity (low flowability) can cause more of the cuttings to remain in suspension and cause undesirable high densities. Gel strength is more an indication of flowability after the slurry has been allowed to set. If the gel strength is too high, the slurry could stiffen after circulation stops and again impede placement of tremie concrete. The contractor proposed to keep the slurry apparent viscosity at or above 7 centipoises when measured at the point of mixing and before discharge to a reserve tank or the trench. The plastic viscosity was proposed to be kept less than or equal to 30 centipoises at the time of concrete placement. The viscosity tests were done with a direct reading viscometer. The 10-minute gel strength was to be kept at or above 2 pounds per 100 square feet in the mixing tank prior to discharge for storage or excavation. This test was also made with a direct reading viscometer. The quick check test for viscosity, the Marsh Funnel, was used frequently and the slurry was required to pass in 30

seconds or more.

The caking ability of the slurry is measured by the filtrate and filter cake test. The filtrate test determines how much slurry is lost through a filter in 30 minutes. High losses indicate a slurry that is incapable of sealing off a pervious face. If the slurry forms a cake on the filter paper (i.e. the trench sidewall) the loss will be low since bentonite is highly impermeable. The thickness of the cake on the filter paper is measured as an indicator of caking ability (how fast a cake will form or reform on the sides of the trench when damaged). The contractor proposed to keep the filtrate loss under 50 cubic centimeters when subjected to 100 psi pressure for 30 minutes. The test was required to be run using a "Whatman" Number 50, S&S Number 576 or equivalent filter.

The chemical environment is measured by the pH. Neutral environments have a pH of 7. Acid environments have a pH less than 7 and alkaline environments have a pH greater than 7. An alkaline environment is desirable, however, if the pH goes over 10.5 the clay particles (bentonite) will tend to clump and settle out. This will cause the density of the slurry to drop and require the addition of more bentonite (at a lower yield of acceptable slurry per bag of bentonite). The contract required the alkalinity of the slurry to be maintained between 8 and 11.

The sand content measures the percentage of sand by volume in a slurry

sample and will be used to determine the cleanliness of the trench prior to tremie concrete placement. High sand content could cause contamination of the concrete or inclusion of sand seams (therefore, water passages) along rock and adjacent panel contact surfaces. The contract required the sand content be less than 1 percent prior to concrete placement.

Organic and inorganic substances could be added (subject to approval) which change the slurry properties. Two types are viscosity reducing agents known as "thinners" and deflocculating agents. Soletanche indicated in their proposal to use sodium bicarbonate or lignosulfonate to control viscosity. Soletanche used sodium bicarbonate almost exclusively to treat the slurry. Lignosulfonate is an organic thinner and a by-product of paper pulp. It causes the clay particles to disperse and lowers the slurry viscosity. Allied Colloids Alchemer 72 is a polymer thinner. Polymers act more quickly than organic compounds. Both thinners are available in liquid form. Sodium bicarbonate (baking soda and in crude form, soda ash) acts as a deflocculent. It is commonly used when high concentrations of calcium or magnesium are present in the soil, groundwater or water source. Calcium can also enter the slurry during excavation of adjacent concrete panels (secondary panel excavations typically removed 4 inches to 14 inches of concrete from the adjacent primary panels) or dental concrete placements along bedrock. The sodium acts to negate the clumping effect of calcium and magnesium and allows the bentonite to retain its ability to swell and absorb water. The same viscosity can therefore be maintained with less bentonite (slurry yield is improved). Allied Colloids

sample and will be used to determine the cleanliness of the trench prior to tremie concrete placement. High sand content could cause contamination of the concrete or inclusion of sand seams (therefore, water passages) along rock and adjacent panel contact surfaces. The contract required the sand content be less than 1 percent prior to concrete placement.

Organic and inorganic substances could be added (subject to approval) which change the slurry properties. Two types are viscosity reducing agents known as "thinners" and deflocculating agents. Soletanche indicated in their proposal to use sodium bicarbonate or lignosulfonate to control viscosity. Soletanche used sodium bicarbonate almost exclusively to treat the slurry. Lignosulfonate is an organic thinner and a by-product of paper pulp. It causes the clay particles to disperse and lowers the slurry viscosity. Allied Colloids Alchemer 72 is a polymer thinner. Polymers act more quickly than organic compounds. Both thinners are available in liquid form. Sodium bicarbonate (baking soda and in crude form, soda ash) acts as a deflocculent. It is commonly used when high concentrations of calcium or magnesium are present in the soil, groundwater or water source. Calcium can also enter the slurry during excavation of adjacent concrete panels (secondary panel excavations typically removed 4 inches to 14 inches of concrete from the adjacent primary panels) or dental concrete placements along bedrock. The sodium acts to negate the clumping effect of calcium and magnesium and allows the bentonite to retain its ability to swell and absorb water. The same viscosity can therefore be maintained with less bentonite (slurry yield is improved). Allied Colloids

The bentonite slurry with the suspended soil particles coming from the trench excavation flows directly onto the coarse shaker screen of the desander. The shaker screen is made of two superimposed screens of differently sized openings. This first operation eliminated all particles larger than 1/4 inch in diameter. The bentonite slurry is then pumped to the hydrocyclone where the sand is separated. This sand is screened and drained through a different shaker screen, then stockpiled in front of the desander in wet, but clean condition. The bentonite slurry passing through the hydrocyclone is then accumulated in a tank. A pump sends the bentonite slurry to a small hydrocyclone, which separates the very fine soil particles. The bentonite slurry is finally sent to the tanks placed under the desanding plant, where it can be used again for the trench excavation.

The slurry that enters the storage tanks can be sampled and tested and treated with chemicals, if necessary. Untreatable slurry is pumped up to the waste storage pond to settle out the bentonite. The sludge is then pumped to the northern wet waste area and disposed.

The contractor maintained 2,110 cy of slurry in storage. There were four storage ponds holding 630 cy each (one was used for holding untreatable waste slurry prior to disposal into the waste areas) and steel tanks below the desanding unit that held 220 cy. A schematic sketch of the slurry flow is provided in figure VI-1.

D. <u>Excavation</u>. The cutoff wall excavation was accomplished primarily by a sophisticated rock mill built by Soletanche, Inc. called the Hyrdofraise. Very few areas were excavated by conventional chisel and grab methods and primarily only where boulders in the Mud Mountain Complex overburden were encountered. The components of the Hydrofraise are illustrated in figure VI-2. The components are housed and attached to an 80 ft. rigid steel frame (3) transported to the site in two boltable sections. The total unit weighs 44 tons. At the bottom end (1) of the frame are two cutter drums and motor assemblies that are built to the same width as the wall to be excavated. Using different size drums, the contractor excavated the Type I wall sections to 32 inch width and the Type II wall section to 40 inch width. The drums are fitted with tungsten carbide tipped cutters. The drums rotate in opposite directions to excavate the soil and bedrock. The speed of rotation could be controlled individually to drive the Hydrofraise and make gradual corrections in direction within the plane of the cutoff wall. The drums were also mounted on a tiltable plate to allow driving the Hydrofraise in and out of the plane of the cutoff wall. A dredge type pump (2), situated just above the drums, pumps the excavated material suspended in slurry from an intake between the drums through about a 6 inch diameter steel pipeline and flexible hose (6) to the desanding station.

A 200 ton Manitowoc 4100 heavy crawler crane with a 170 foot boom was used to support and manipulate the Hydrofraise by way of a crane-operated cable. There is always a crane operator and Hydrofraise operator in individual cabs to

run the operation. The Hydrofraise operator cab is attached just outside the crane operator's cab so that they sit essentially side by side. The Hydrofraise operator controls the cutter drums, dredge pump, clean slurry return pumps at the densanding station and a hydraulic feed cylinder (7) resembling a Kelly Bar which allows (within a range of about 10 to 15 feet) the Hydrofraise to be advanced at a constant rate or maintain a constant weight on the cutter drums.

Mounted behind the crane is a 750 horsepower (at 2000 rpm) diesel power pack (5), manufactured by Caterpillar Inc., which supplied hydraulic power through hoses (9) to the three down-the-hole motors. Two of them drive the cutter drums and the third drives the dredge pump. The dredge pump is able to circulate 1,980 gpm of slurry at a maximum delivery pressure of 100 psi. The cutter drum motors are designed to produce high torque at low speeds of rotation (about 16 rpm). The Hydrofraise used on this project was the latest version designed for great depths and excavation in hard formations. The torque at the cutter drum was increased from 27,500 foot-pounds on previous units to 81,400 foot-pounds on the latest unit. The hydraulic motor produces 270 horsepower. The system was designed to excavate down to a maximum depth of 415 feet.

The Hydrofraise had electronically operated controls and was fully instrumented with digital, as well as, strip chart recorders. There were four sources of data for quality assurance inspectors to monitor progress: the

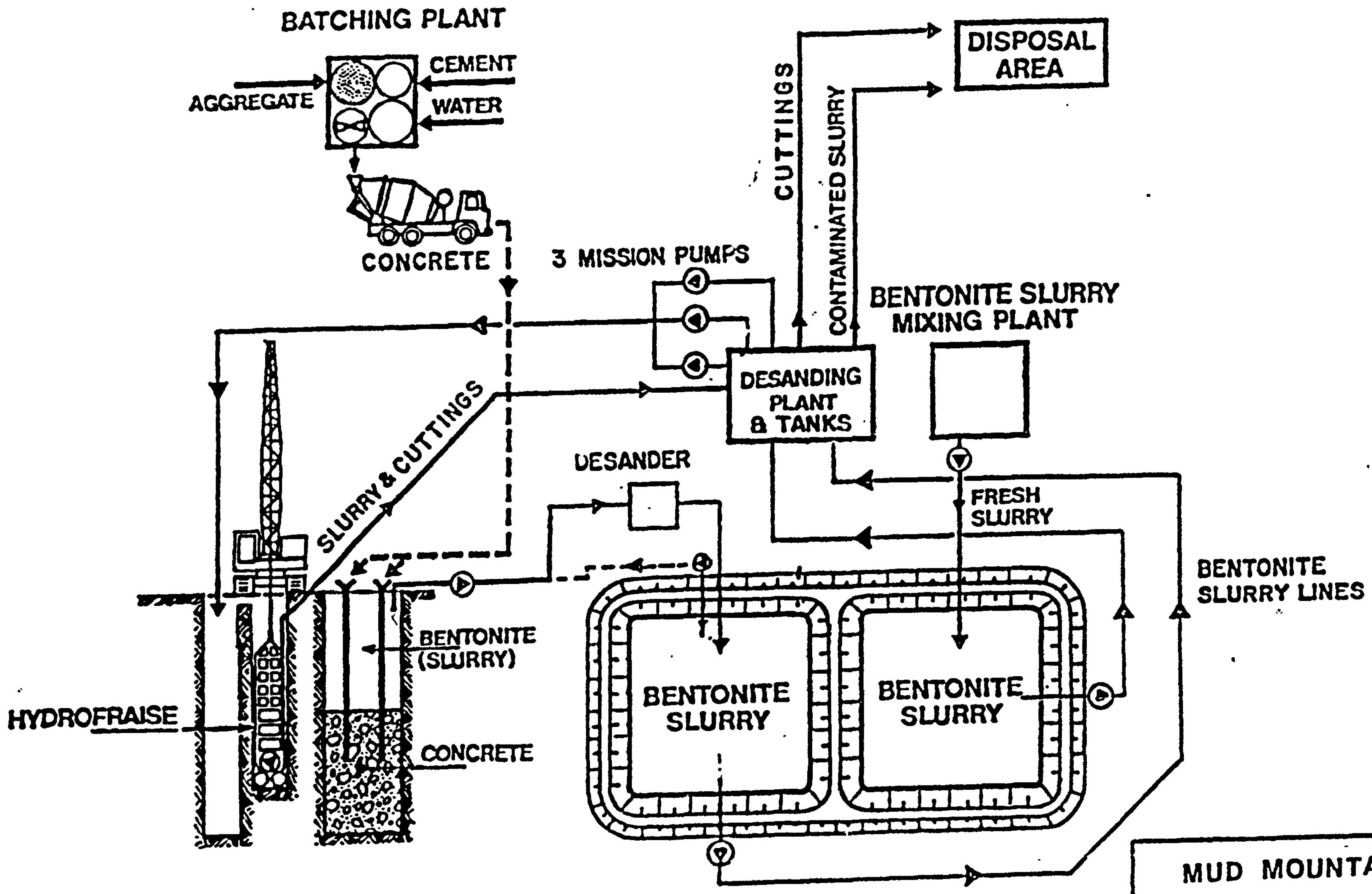
Hydrofraise operator's dashboard digital readouts on depth and deviation; a computer screen on the upper corner of the dashboard displaying depth, torque, deviation, and time; a strip chart recorder on the outside wall of the Hydrofraise cab displaying depth and deviation; and a data compiler on the side of the crane displaying time in hours, minutes and seconds, depth in centimeters and other data. The compiler records information every time the Hydrofraise advances 5 centimeters down the trench. When the Hydrofraise pulls out, the compiler stops recording data, but continues to display the last set of data. This was very useful in carrying out production studies on the Hydrofraise.

The Hydrofraise power pack is equipped with an early warning system that detects intrusion of slurry into the hydraulic lines supplying the down-the-hole motors. A horn goes off when a critical level of bentonite is detected in the hydraulic fluid. Normally, proper changing of hydraulic oil filters were sufficient to prevent the build-up of slurry in the hydraulic lines to levels critical enough to shut down the operation. Considerable downtime is involved in purging the system once the slurry levels in the hydraulic oil exceed certain limits.

Excavation rates averaged about 89 square feet per hour in the core material of the dam and about 16 square feet per hour in the andesite bedrock. Pick consumption averaged 55 picks per 10 hour shift to 60 picks per 11 hour shift. Total pick consumption for the Type I wall excavation was about 7,500

picks and 10,000 in the Type II wall. Detailed production data and downtime records for panels 124, 129, 139, 122, 126, 141, 120, 135, 130, 140, 134, 128, 138, 136, and 142 are included in the appendix to this report.

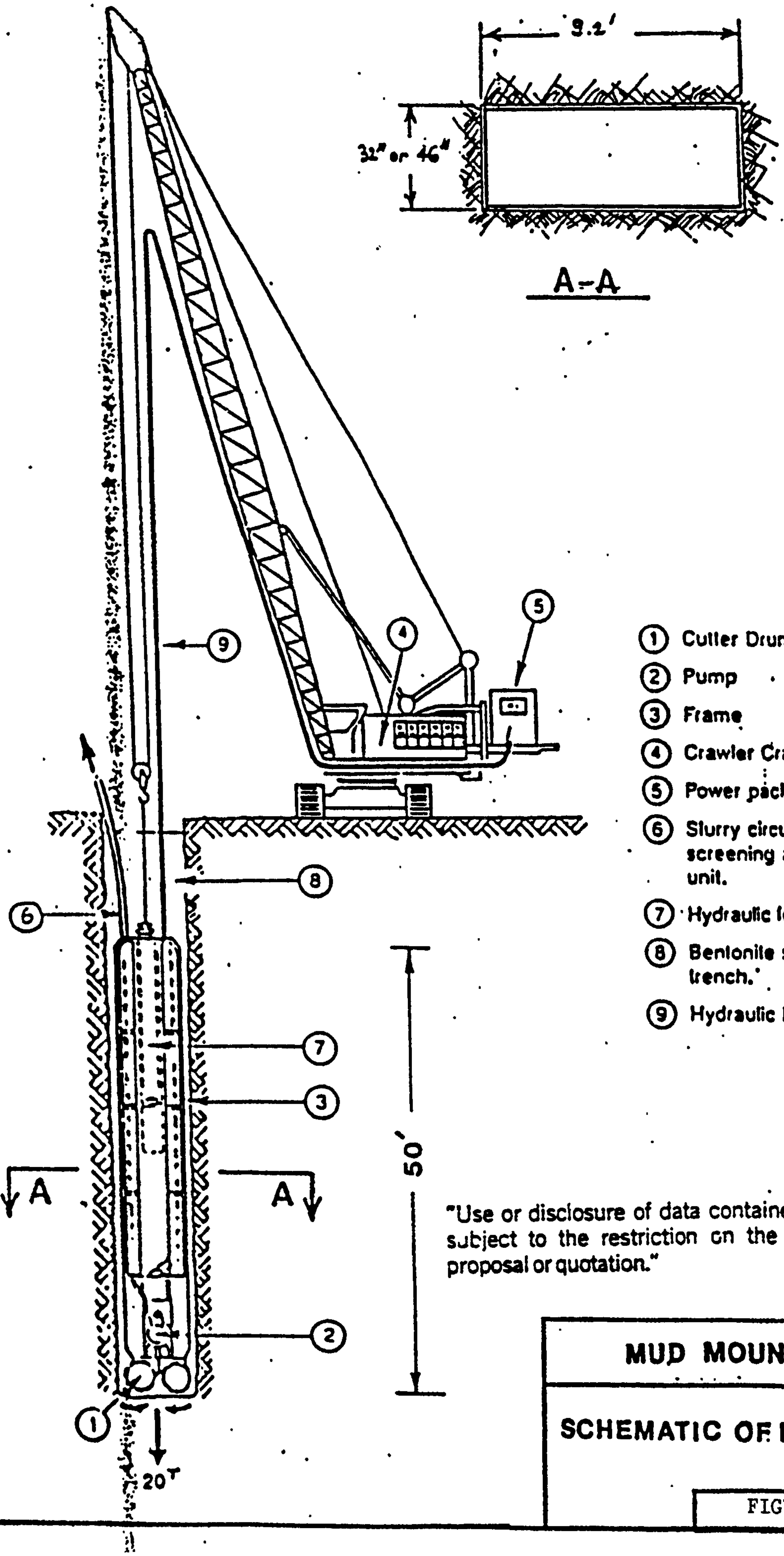
Prior to placement of concrete, the panel was desanded by using the Hydrofraise to act as a sump pump and clean the panel bottom, as well as the slurry in the trench. The specification required the panel slurry to be cleaned to a sand content less than or equal to 1 percent. This was deemed unreasonable or unnecessary by the contractors submitting proposals during the bid process, however, the Hydrofraise equipment could routinely achieve sand contents less than 1 percent prior to concrete placement. Detailed information on the equipment, methods, mix designs, and quantities involved in the concrete placements are discussed in Section XII of this report.



MUD MOUNTAIN DAM

SLURRY FLOW CHART

FIGURE VI-1



① Cutter Drums
② Pump
③ Frame
④ Crawler Crane
⑤ Power pack
⑥ Slurry circulation hose to screening and desanding unit.
⑦ Hydraulic feed cylinder
⑧ Bentonite slurry filled trench.
⑨ Hydraulic hoses

"Use or disclosure of data contained on this sheet is subject to the restriction on the title page of this proposal or quotation."

MUD MOUNTAIN DAM

SCHEMATIC OF HYDROFRAISE

FIGURE VI-2

SECTION VII

QA/QC PROGRAM

SECTION VII - QA/QC Program

Key objectives of the QA/QC program were aimed at addressing design concerns about ensuring verticality of excavations, proper overlap and watertightness between panels, trench stability, proper embedment of panels into bedrock, high quality homogeneous well-consolidated concrete, and proper concrete strength gain. Some concerns cited by Bureau of Reclamation engineers working on the Navajo Dam project in Farmington, New Mexico related to a lack of redundancy in verifying the contractors panel verticality and continuity assessments. Another concern cited was the possibility that native soils mixed with water may have been used periodically at Navajo Dam in lieu of a bentonite slurry mix. The "Hydrofraise" excavating machine brought to the Mud Mountain Dam project was the latest version incorporating many new instrumentation and hardware features which, in addition to the tailored QA/QC program, addressed these design and construction concerns.

The original contract plan required steel guide members to be installed in "primary" panels on each side of subsequent "secondary" panel excavation, see figure VII-1. The steel primary guides were to incorporate an inclinometer casing to verify the verticali'y of the installation. The secondary excavation was required to be carried out while the excavator remained in constant contact with each adjacent primary guide to ensure verticality and continuity of the

cutoff wall panels. An option was provided to run a template down after the secondary excavation in contact with each adjacent primary guide to verify the continuity and verticality of the panels before concrete placement.

The contractor proposed to eliminate the primary guides and utilize the constant read-out/feedback capabilities of the hydrofraise to ensure verticality and continuity of the panels. Room for some deviation was to be accounted for primarily by increasing the excavated thickness of the deeper panels to ensure at least a 24 inch thick continuous wall was constructed. This Value Engineering Contractor Proposal was accepted providing instant contract savings of $1,202,399.00 with the Government realizing a savings of $613,391.00.

The VECP deleted the steel truss primary guides proposed under the original contract to be installed in "single bite" panels (panels excavated by a single pass of the Hydrofraise) and called instead for construction of three bite primary panels with the middle bite everlapping and going in between the first and second bites, see figure VII-1. The Hydrofraise was to be lowered down between the bites after excavation was complete to assure that no unexcavated portions remained between; thus ensuring continuity of the primary panel. The secondary panel was excavated between the adjacent primary panels in an alternating pattern.

The VECP was not accepted without redundant verticality/continuity checks.

The contractor proposed to dye the lower 20 feet of the primary panels in alternate color schemes of black and red concrete as a qualitative check on continuity and verticality (see figure VII-2). The primary panels were spaced such that the Hydrofraise would overbite into the adjacent primary panels when excavating the secondary. QA/QC personnel could then verify this at the lower depths of excavation by retrieving colored concrete pieces from the desanding operation. This in itself gave little assurance that the overlap was at least 24 inches throughout the wall. The contractor believed the 24 inch overlap could be assured by the assumption thac the concrete pieces retrieved were broken off portions of the "key" left on each excavated primary joint between the two cutting heads of the Hydrofraise, see figure VII-3. In reality, the pieces did not resemble in any way a neat key and so the method in itself cannot be used as a QA/QC tool.

The coloring did come into play under another redundant scheme using an 80 foot long drilling guide which duplicated the dimensions of the Hydrofraise down in the trench. The so-called "fish" (see figure VII-4) could be positioned at any depth of the secondary panel. Once set, a coring rig could send an "AX" or "EX" core barrel and drill string down the tubing and sample from the face of each adjacent primary panel joint. Retrieval of four cores, two from each primary interface, geometrically assured that at least a 24 inch overlap existed at that location of the panel. This redundant check was instrumental in detecting an initial error in the hydrofraise's built-in inclinometer. The fish was first used in Panel 16. It cored two samples of

black concrete from Panel 15, which was correct and verified both minimum width of the wall and closure. However, on the other side (Panel 17) it cored approximately 6 inches of andesite bedrock before going into pink concrete. Having occurred in both holes on this side, it confirmed minimum panel width, but not closure. This initiated a calibration check of hydrofraise's inclinometers, which had been (permanently) set after assembly. The verticality of the hanging hydrofraise was checked with a theodolite. It was found that the hydrofraise's inclinometer sensor mounting bracket, located inside the open framework, had been bent by a large cobble or boulder falling out of the excavation wall and onto the mounting bracket during excavation. After this incident, Panel 16 was trimmed along the interface with Panel 17 and rechecked with the fish. It then became standard practice to verify the accuracy of the hydrofraise inclinometers with the theodolite before excavation of each panel.

The verticality data from the primary panel excavations could be compared while or before the secondary panel between them was excavated. The actual profile of the panels could be overlayed to find the location where continuity could be most questionable. When the secondary excavation was complete, the profile of the secondary panel could be generated to choose the depth at which the "fish" should be set to verify continuity. In most instances, setting the fish at the lowest depth was used to verify continuity. The Hydrofraise was found to be a highly reliable, accurately instrumented excavator.

Another redundant check was added later after the contractor experienced heavy slurry losses and trench instability. The primary panel excavations were reduced back to single bites to reduce the time that panels remained open. This increased the number of secondary panels and panel joints. The contractor was therefore tasked with meeting tight tolerances on more occasions and cited reduced flexibility in not being able to make up for tolerance errors within the three bite primary panels. The contractor proposed to increase the overlap of the secondary panels into the primary panels and provide additional verticality checks. The contractor devised a well conceived, yet simple, "wear test." The wear test consisted of periodically installing excavator teeth without carbide tips onto the Hydrofraise cutter heads then after excavating short reaches, checking the teeth for characteristic wear patterns. If a band of teeth over at least a 24-inch width showed the characteristic wear pattern from excavating concrete in adjacent primary panels, the Hydrofraise was considered to be within acceptable verticality limits and assured continuity within that reach. These checks were done at approximately 80 ft. increments because the frame of the Hydrofraise is 80 ft. and deviations can only occur in these 80 ft. reaches.

The bedrock embedment of the panels was to be computed by straight line interpolation between bedrock contact depths from core drilling information in open primary panel trenches. This method was waived in favor of using torque readouts from the Hydrofraise instruments to pinpoint depths of contact with bedrock. This was deemed more accurate because drill wander concerns were eliminated and each cutter head provided a contact point so two bedrock depths were obtained. Another benefit was the reduction in time that a primary panel

remained open. Drilling in open panels could have gone on for days at a time. This was deemed risky in light of the slurry loss problems encountered.

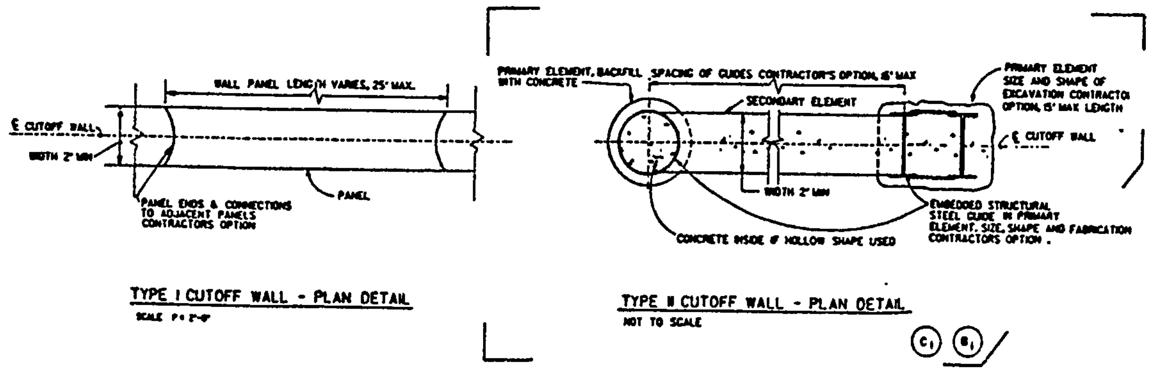
Slurry tests were made on a daily basis. The slurry mixing station was readily accessible and in general view to assure properly mixed slurry was maintained in the storage ponds nearby. Slurry tests were made from the trench, from the pumps, and from the storage ponds. No irregularities were found in the quality, production, or maintenance of the slurry supply (see figure VII-5).

The concrete QA/QC program is covered separately in this report. Key parameters checked were slump, air content, tremie embedment in fresh concrete, placement rate, go-devil performance and strength gain to ensure the concrete would flow properly, consolidate well, remain homogeneous and uncontaminated and achieved proper strength for adjacent excavation and future performance. Concrete coring was carried out after curing to verify the quality of the concrete. Some honeycombed areas were found in earlier panels, but pressure tests indicated they were not continuous through the panel. Nevertheless, the core holes were pressure grouted and intermittant corings were made and pressure grouted as well. Maintaining the concrete slump in the specified range of 7 to 9 inches and a slight change in the mix proportions cured the problem with honeycombing. Quality Control checks using core holes were correspondingly relaxed later on. The drilling subcontractor, Boyles Brothers of Salt Lake City, Utah had relatively little difficulty maintaining the core

hole in the panel even to 400 plus foot depths. An Eastman apparatus was used along with a steerable downhole destructive drilling technique (Navidrill) to correct non-vertical drilling. Core recovery throughout the job was excellent.

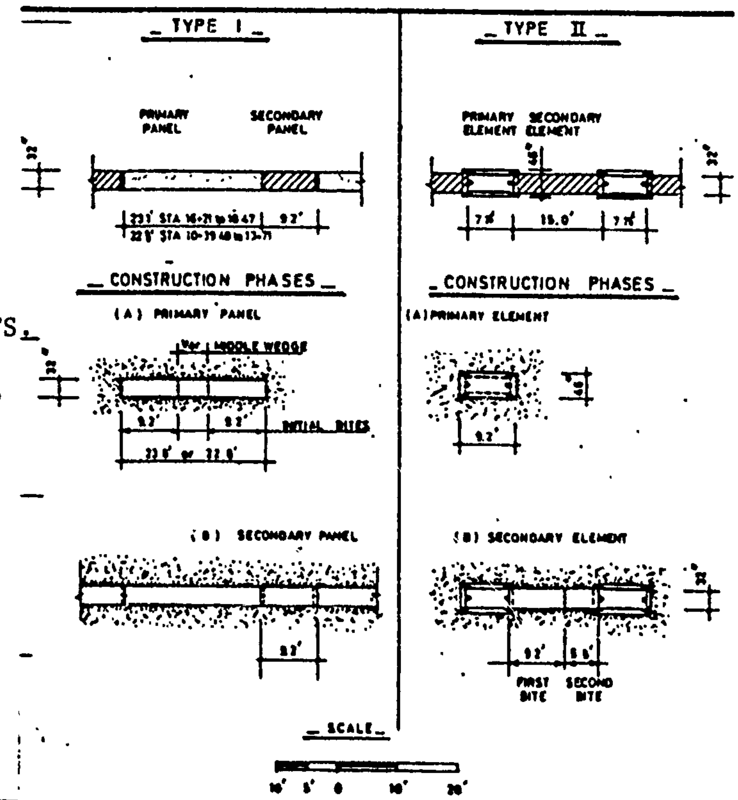
Quality Control Reports were received generally in two days. This was because the contractor worked double shifts which ended at 0400 hrs or during around the clock operations at 0700 hrs. The QCR was provided about a day after the end of the night shift. The contractor utilized serialized Extra Work Reports to document work considered outside contract requirements. This facilitated response on potential claim items and most were resolved without time or cost impact.

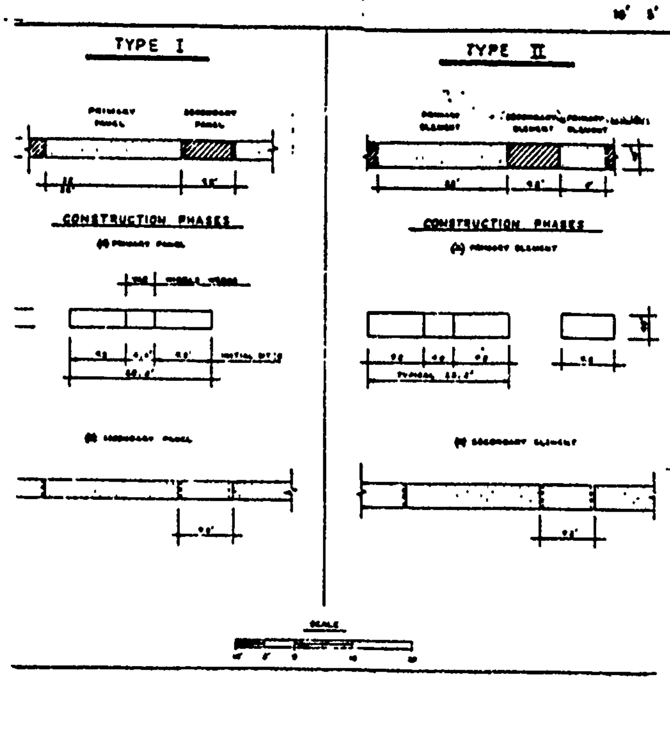
CHRONOLOGY OF PANEL CONSTRUCTION TECHNIQUE



CONTRACT PLANS: STEEL GUIDE MEMBERS REQUIRED IN PRIMARIES TO GUIDE SECONDARY EXCAVATION.

CONTRACTOR'S ORIGINAL PROPOSAL: RECTANGULAR STEEL TRUSS GUIDE IN PRIMARY ELEMENTS. TWO WELDED STEEL PIPES INTEGRAL TO TRUSS ALLOWS INSTALLATION OF INCLINOMETER TUBING FOR VERTICALITY CHECK.

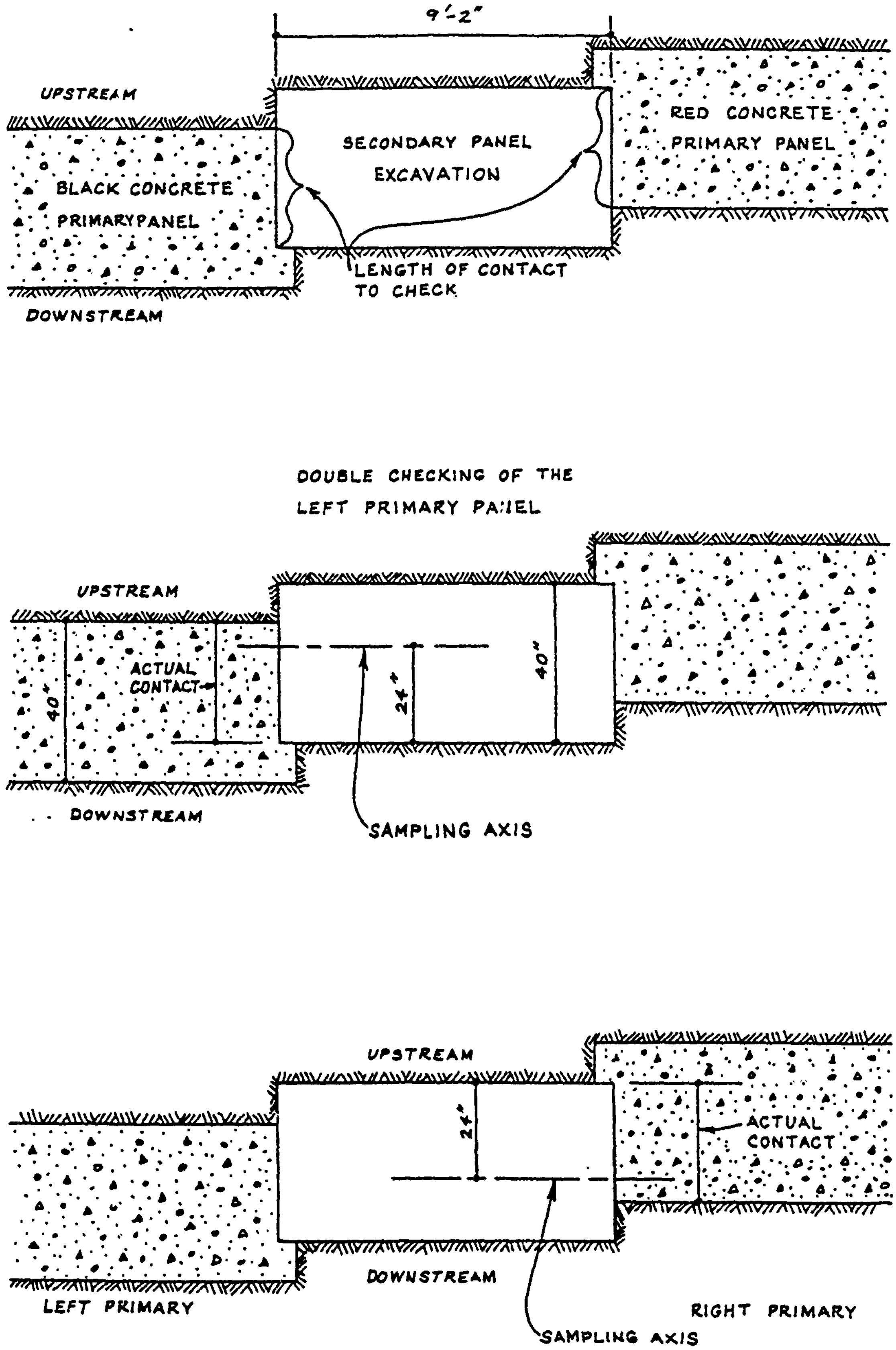




CONTRACTOR'S VALUE ENGINEERING CHANGE PROPOSAL: STEEL GUIDE MEMBERS DELETED. SECONDARY SHORTENED TO SINGLE EXCAVATION PASS DIMENSION OF HYDROFRAISE EXCAVATOR. SOME PRIMARY PANELS INCREASED IN LENGTH TO REQUIRE THREE EXCAVATION PASSES, THE MIDDLE BEING REMOVED LAST (THE WEDGE).

FIGURE VII-1

DOUBLE CHECKING OF CAST IN PLACE WALL CONTINUITY

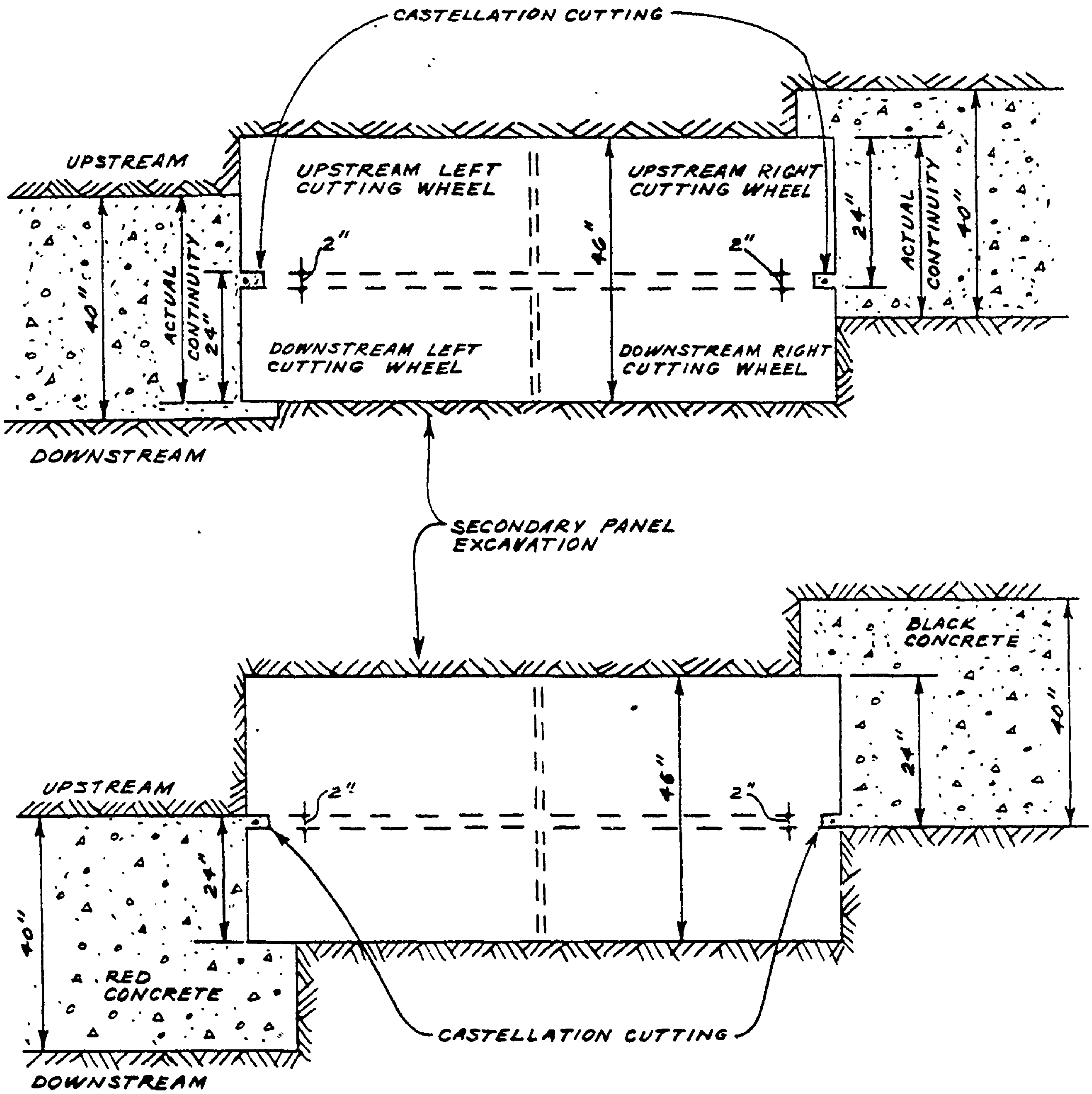


NOTE:

THIS CONTROL DOUBLE CHECKS THE DATA OF THE EMPAFRAISE REGARDING THE CONTINUITY OF THE WALL.

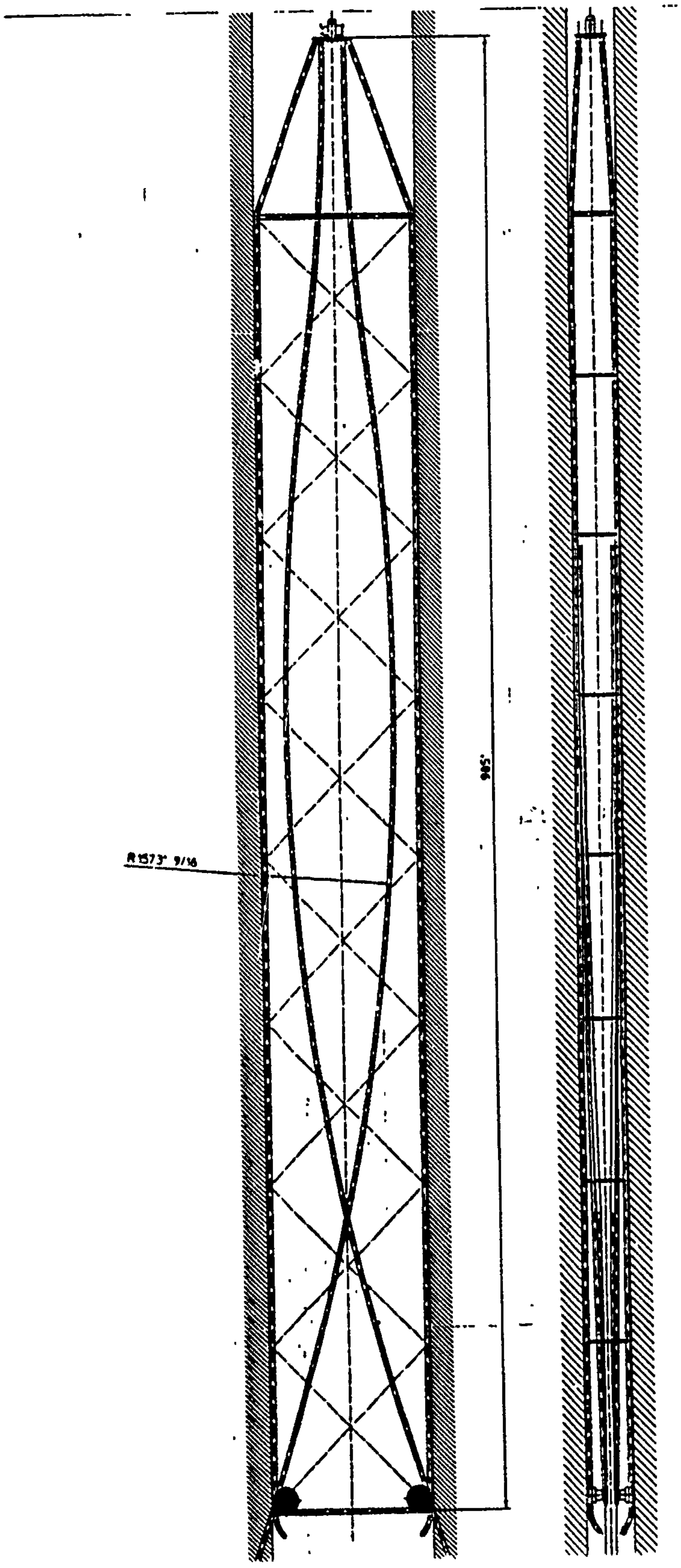
FIGURE VII-2

CONTINUOUS CHECK BY SAMPLING CASTELLATION CUTTINGS *

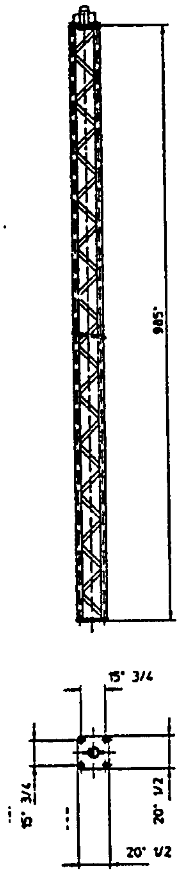


* FOOTNOTE: CASTELLATION CUTTINGS ORIGINATE FROM THE CENTRAL RIDGE LEFT IN CONCRETE BY THE HYDROFRAISE.

FIGURE VII-3



DRILL FRAME ("FISH")
TUBING WITH LONG RADIUS BENDS
GUIDES CORE BARREL AND RODS



"DRILL STEM"

DRILL FRAME AND STEM
POSITIONED IN TRENCH

FIGURE VII-4

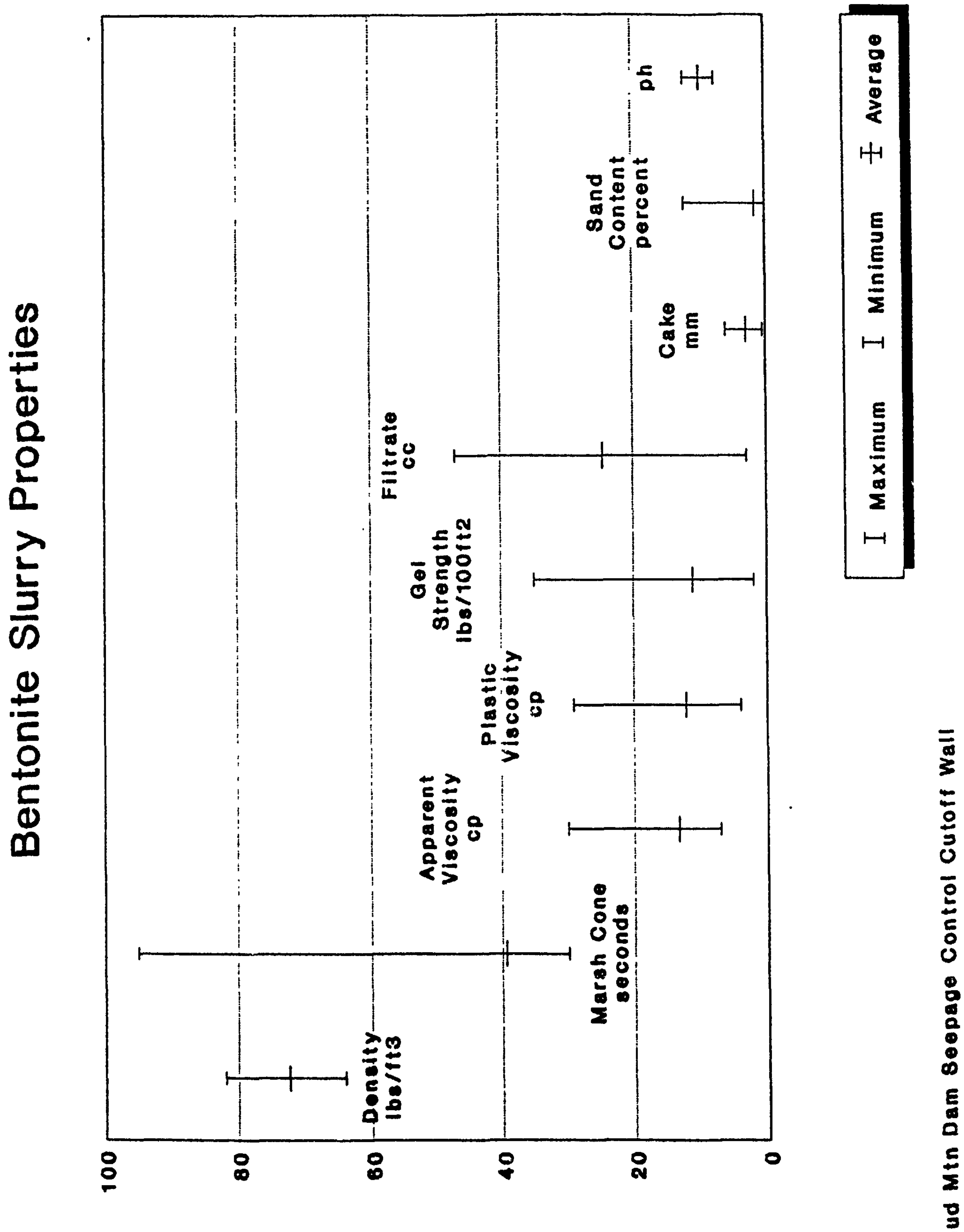

Bentonite Slurry Properties
100
80
60
40
20
0
Density lbs/ft3
Marsh Cone seconds
Apparent Viscosity cp
Plastic Viscosity cp
Gel Strength lbs/100ft2
Filtrate cc
Cake mm
Sand Content percent
ph
Maximum
Minimum
Average
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE VII-5

SECTION VIII

CONTRACT CONSTRUCTION PROBLEMS

SECTION VIII - CONTRACT CONSTRUCTION PROBLEMS

Several incidents of slurry losses through hydrofracture of the embankment through preexisting or induced cracks impacted progress on the slurry wall construction during the first four months. Though the contract was well prepared and procured to allow the contractor to prepare for potential construction problems in advance, the regular occurrence and severity of the slurry losses were not anticipated. The contractor was required to submit an emergency plan as part of the technical proposal for dealing with exceptional slurry losses. Soletanche had outlined a four step emergency plan.

Step one was to be implemented if the three "mission" pumps feeding slurry to the trench at a combined rate of up to 2,000 gallons per minute were not capable of maintaining the slurry level in the trench. The excavation would be stopped and the normal circulation would be continued allowing the hydrofraise operator to control a slurry loss of 2 to 3 feet per minute or 690 gallons per minute.

Step two would be implemented if the slurry level continued to drop after implementation of step one. The cutter wheels would be restarted to generate cuttings for plugging leaks, but the slurry return from the trench would be stopped. The hydrofraise operator then could use the full 2,000 gallons per minute capacity of the mission pumps to maintain the trench slurry level. This would allow control of slurry losses of up to 10 feet per minute. This step

could be theoretically implemented for about 1.7 hours with a 1,000 cubic yard slurry supply. The contractor had proposed to provide this storage capacity by building slurry ponds holding 800 cy and holding tanks storing another 200 cy. Actually the contractor had four slurry ponds holding 630 cy each, but one of these was dedicated for waste slurry storage prior to disposal. In addition, The desanding unit could store 220 cy of slurry in steel tanks below the cyclones and shaker screens. Therefore, the total onsite storage capacity was about 2,110 cy.

Step three involved removal of the hydrofraise from the trench while continuing to maintain the flow of slurry into the trench to keep the slurry level as close to the top of the excavation as possible. The trench would then be backfilled with sand and/or graded gravel to plug the point of leakage. After implementing step three, re-excavation of the panel would only begin upon the approval of the Contracting Officer.

Re-excavation of a backfilled panel under step four of the emergency slurry loss plan would require a supply of premixed grout to be available for injection into the trench through a 6 inch pipe using the hydrofraise slurry in the event of another slurry loss. Grouting would only begin on the order of the Contracting Officer's Representative.

In practice, step two was implemented briefly or completely bypassed and step three was immediately implemented after step one failed to stop the slurry

loss. Step four was implemented only twice and then abandoned in favor of dumping pallets of bagged cement into the trench mixing then sending the hydrofraise in ᴗ mix it in with the core material cuttings. The application of step four resulted in time consuming difficulties during the re-excavation of such panels. The thick mixture of cement, bentonite, and soil would plug the slurry return pump, gum up the cutter wheels and require extensive treatment of the slurry at the desander with sodium bicarbonate to maintain acceptable slurry consistency. Continuous or routine application of such methods would have resulted in unacceptable progress.

The work was started on a day shift basis and then went to two 10-hour shifts beginning on June 5, 1989. Cutoff wall construction began on the evening of May 16, 1989 with the excavation of panel 15 bite A (15A). The excavation was carried to a depth of 24 ft. between 1600 and 1900 hours. The next day a slurry loss occurred sometime after noon during a one hour lunch break when the hydrofraise was out of the panel and idle. The panel had been excavated to a depth of 39 ft. and the slurry level had dropped down 17 ft. and stabilized. The slurry level was brought back up to within 4 ft. of the work platform to measure a loss rate of 16 inches per minute or 320 gallons per minute. Sand was used to slow the leakage to 6.7 inches per minute or 135 gallons per minute. Additional sand and gravel backfill and the addition of 500 pounds of bentonite powder temporarily brought the slurry loss under control. Minutes later the slurry loss resumed and after sand and gravel backfill had been added up to within 13 ft. of the work platform the slurry

loss finally stabilized. The contractor estimated that over 90,000 gallons of slurry had been lost.

Excavation work resumed on May 18, 1989 in panel 15C. At a depth of 28 ft., the three "mission" pumps supplying slurry to the trench could not maintain the slurry level. Each mission pump is capable of supplying about 675 gallons per minute, therefore, slurry was being lost at over 1,980 gallons per minute. Excavation was stopped to measure the slurry loss at 19 inches per minute or 362 gallons per minute. The addition of sand and gravel backfill reduced the slurry loss to 310 gallons per minute. Backfilling the trench up to within 12 ft. of the work platform stopped the slurry loss. The contractor estimated 81,000 gallons of slurry were lost.

The contractor moved to panel 17C and began excavation on May 19, 1989. Slurry began to be lost slowly at a depth of 27 feet. The loss increased to a rate of 14 inches per minute or 285 gallons per minute at a depth of 31 feet. The trench was backfilled to within 15 ft. of the work platform and stabilized, however, in the course of backfilling panel 17C, panel 15C suddenly lost slurry in what the shift superintendent described as a vortex into a 20 inch hole on the downstream face of the panel. The contractor was directed to advance split spoon driven exploratory holes around panels 15 and 17 to find potential avenues of slurry loss. This exploratory work on May 20 and 21, 1989 revealed nothing of significance. Some thought had been given to the possibility that the transition zone of 6 inch rock spalls intercepted the trench, but this was

proved incorrect.

The contractor returned to re-excavate the previously backfilled panel 15A on May 22, 1989. At the Government's direction, the contractor was prepared to implement step four of the exceptional slurry loss action plan. The hydrofraise was fitted with an auxiliary grout tube to allow injection of cement-bentonite grout with sodium silicate admixture for accelerated set at the point of excavation. The excavation in panel 15A reached a depth of 33 ft. when a slurry loss of 5 inches per minute or 90 gallons per minute began. Slurry in panel 15C remained stable since it was almost completely backfilled. The contractor immediately began injecting premixed grout into the excavation and reduced the loss to 3 inches per hour or 57 gallons per minute, but it rapidly increased back to 5 inches per minute. The hydrofraise was then pulled up 3 ft. while continuing to inject grout. After injecting a total of 23 cubic yards of grout and using 6 barrels of sodium silicate, the slurry loss was stabilized. The contractor was required to establish 24 hour watch on the slurry levels in open excavations.

The contractor returned to re-excavate panel 15C on May 23, 1989 and was prepared to grout under step four of the slurry loss procedures. The excavation went beyond the previous slurry loss depth of 28 feet. The 6 cy of premixed grout was then wasted. When the hydrofraise reached a depth of 74 ft. on May 24, 1989, a major slurry loss occurred in panel 15C. There was approximately 420 cy of slurry lost. The hydrofraise was removed and the

trench was backfilled with sand. During the excavation, the contractor encountered problems with thickened slurry. This was probably due to intrusion of cement treated slurry from adjacent panels. At this point the contractor cited a changed condition.

Re-excavation of panel 15A and 15C began again on May 30, 1989. Minor controllable slurry losses occurred, but problems were experienced with grout thickened slurry gumming up the pumps and cutter heads. All three bites of panel 15 were excavated without further incident. The panel was concreted on June 7, 1989.

Late in May 1989 meetings were held with the contractor and internally with Seattle District and North Pacific Division to develop a plan of action and reduce the slurry losses. An investigative drilling and grouting program was initiated on June 7, 1989 under a contract modification. The program involved drilling 8 inch diameter rotary drill holes using a Schramm T685H truck mounted drill. The holes would be drilled using the same bentonite slurry used in the cutoff wall excavation. The slurry level in the hole was to be measured every 30 ft. in depth for five minutes. Premixed grout was to be injected under gravity feed through the drill string in the event of a slurry loss. Sodium silicate would be added as a stiffening agent if the losses continued in the hole. The holes were drilled in advance of the hydrofraise to pretreat potential slurry loss zones. Between June 7, 1989 and July 17, 1989, twenty-two locations were drilled, but no correlation could be made between slurry

losses or the lack of them in the drill holes and the occurrence of slurry losses in the same areas later with the hydrofraise. In the 5,053 linear feet of drill holes, a total of 292 cy of grout was used in seven of the holes. See table VIII-1.

On June 15, 1989 a slurry loss occurred in panel 13C at a depth of 93.5 ft. while the hydrofraise was being withdrawn from the trench. The trench was backfilled with sand under step three of the slurry loss procedures. Except for a controllable loss on June 20, 1989 the panel was re-excavated without further incident between June 19, 1989 and June 22, 1989.

It should be pointed out that borehole camera investigations in concrete core holes in panel 15 and 17 between July 12 and 13, 1989 revealed water draining out from a pinhole stream in panel 17. The possibility that a subsurface flow from Lower Cascade Creek running along the bedrock/Mud Mountain Complex was considered. Later a concrete coring in panel 9 revealed washed aggregates at the contract between the bottom of the panel and bedrock. This was considered possible evidence of such a subsurface flow washing out the green concrete at the bedrock contact. This was also considered as a possible cause of the slurry loss in panel 13 and the source of water for the slurry that eventually showed in a spring on the upstream left canyon wall.

A major slurry loss occurred during the early morning hours of June 17, 1989 in panel 35 at a depth of 115 feet. Vertical cracks were noted in the

trench by observers. The cracking appeared to be longitudinal with the axis of the dam. The entire panel lost slurry and there was great concern with collapse of the trench sidewalls. Approximately 30 sacks of bagged cement were dropped into the trench before backfilling the trench with sand. The leak was stopped when the trench had been backfilled to 75 feet. The contractor estimated that 1,050 cy of slurry was lost.

Another large loss occurred in panel 43A during the night shift on June 22, 1989 at a depth of 88 feet. About 800 cy of slurry was lost. The losses apparently occurred over a span of time eventually depleting the storage capacity of the desanding unit tanks (220 cy). Additional supply from the storage ponds in the staging areas was tapped while the hydrofraise was pulled out and backfilling operations began. The drop in the slurry level measured by Government personnel ranged between 2 and 10 inches per minute.

The Project Operations personnel noted a spring of dirty water exiting from the left canyon wall approximately 1,200 ft. upstream of the cutoff wall axis on June 28, 1989. The water was sampled and sent to the North Pacific Division Laboratory in Troutdale, Oregon for analysis. The water was found to contain bentonite. The source of the bentonite was certainly from the slurry losses encountered to that point, but which particular one or ones could not be determined. It was postulated that the slurry was being lost upstream into the rock shell of the dam, but was unable to escape due to silt buildup over the toe of the dam. When the total slurry lost into the rock shell over-topped the

silt level the slurry was able to escape and show up in the creek. The spring was most definitely exiting the rock and this fissure may have been connected to a dammed up reservoir of slurry in the upstream rock shell. Another possibility was that the slurry was attributable to losses in panel 13C two weeks earlier. The time difference could be attributable to travel time through fissures in the rock. Supporting this was the concrete coring in panel 9 indicating a washed zone of concrete at the base of the panel. The possibility that underground flow from Lower Cascade Creek along the bedrock contact with the Mud Mountain Complex had washed the paste from the green concrete at the base of the wall could also have intercepted panel 13 and be interconnected with the dirty spring observed upstream of the dam.

During the early morning hours of June 29, 1989, a very significant slurry loss occurred in panel 39C at a depth of 127 feet. Within minutes cracking was noted at each end of the panel along the edge of the downstream guidewall on top of the dam. The cracks continued to propagate across the top of the dam from panel 39 to panel 17 over 250 feet. The loss rate was not great (between 6 and 10 inches per minute) but continuous and difficult to stop. The slurry loss consumed 1,500 cy of slurry and required 65 bags of cement and 150 cy of sand and gravel backfill before coming under control.

On the afternoon of Saturday, July 8, 1989 during re-excavation of panel 35, a slurry loss occurred at a depth of 106 feet. Even after dumping 34 sacks of cement and lowering the hydrofraise into the trench to mix the cement with

cuttings the slurry was being lost at a rate of 12 inches per minute (230 gallons per minute). Preparations were made to implement step four with 10 batches of grout 1.2 cy each with 1 barrel of sodium silicate accelerator. The down the trench grouting was done using the hydrofraise. There was 150 cy of slurry lost in the panel. The contractor's quality control engineer saw the end wall of the trench excavation crack open about 1/8 inch. The contractor placed fluorescein dye into the slurry to trace the loss.

On July 13, 1989 the contractor attempted one of the deeper panels near the right canyon wall, panel 25. The panel ws excavated to a depth of 90 ft. before slurry was lost at a rate of 32 inches per minute (613 gallons per minute). The hydrofraise was pulled out and backfilling began with 90 bags of cement followed by 66 cy of sand and gravel. Fluorescein dye was also placed in the trench. A vertical crack on the south side of the panel was observed by Corps and contractor personnel. The contractor estimated 470 cy of slurry was lost. The next day the contractor was instructed not to leave panels open to core the foundation bedrock as required by contract. The concrete core holes would be extended selectively into bedrock to obtain cores and water pressure test data.

A meeting was held on July 18, 1989 with geotechnical experts from Office of the Chief of Engineers (OCE), North Pacific Division and the Seattle District to assess the dam safety issues and develop a plan of action. It was agreed that the cracking caused by the slurry losses did not endanger the dam

and the consequences of any internal damage would be mitigated by completion of the cutoff wall. Another attempt in panel 25 or 31 was advisable to determine if the cutoff wall could be constructed of single bite panels with acceptable losses of slurry and in production. Single bite panels would reduce the zone of influence exerted by the hydrostatic pressure of the slurry in the trench and reduce the time of open trench exposure. The time between panel excavation and concrete placement could be reduced by two thirds compared to a three bite panel.

The contractor was directed back into panel 25 and began re-excavation on July 19, 1989. Another slurry loss occurred at 113.8 ft. and the panel was backfilled with 105 cy of sand and gravel. The contractor estimated 700 cy of slurry was lost. The maximum loss rate recorded was over 3 ft. per minute or over 690 gallons per minute. The upper 30 ft. of the trench was exposed. The contractor raised concerns with sub-vertical cracks daylighting into the work platform allowing triangular slabs of the sidewall and work platform to fall into the trench. The workmen became wary as well.

Earlier in the day a concrete placement on panel 37 was terminated after 176 cy had been placed out of 329 cy projected for the panel. The contractor had placed 5 truckloads of very low slump concrete (between 4.5 and 6.5 inches) then experienced problems resetting the tremie pipe. The pipe would not go down after repeated attempts to reseat it. The pipe would stop abruptly with a loud noise as if something solid was in the way. The excavation in panel 25

was only 35 ft. down at the time. Whether or not excavation in panel 25 influenced this incident is not known, but the contractor postulated that the sidewall of the excavation in panel 37 fell in and dropped boulders into the green concrete. The removal of all the concrete placement was started immediately by clamshell, but no evidence of a cave-in was found, however, the panel required 400 cy to complete (21 percent above neatline) indicating a cave-in may have occurred. Clearly the stiff loads of concrete were not responsible for the obstruction. Excavation of the hardened concrete by the Hydrofraise indicated something very hard mixed in the concrete. It was assumed that this confirmed a cave-in of boulders into the trench during the concrete placement.

An attempt was made in panel 119 (originally numbered 19) on July 26, 1989 to see if completed placements of panel 15, 16, and 17 had healed problem areas on the south canyon side. A slurry loss of about 7 inches per minute occurred at 52 ft. consuming 310 cy of slurry and requiring 82 cy of sand and gravel backfill. The upper 12 ft. of the trench was exposed. Thirty minutes after the slurry loss, the Corps representative noticed cracking along the upstream side of the guidewall between panel 119 and panel 143 on the other side of the canyon.

Early the next morning on July 27, 1989 excavation began in panel 143. A crack opened up on the surface of the dam along the downstream side of the guidewall then on the upstream side minutes later. Over a 20 minute period the

cracks continued to propagate and open up a 1/4 inch. This began when the excavation was down 61 ft. and continued down to 102 feet. At 111.5 ft. the trench began to lose slurry at a rate of 17 1/2 inches per minute or 335 gallons per minute. The trench was backfilled with 200 cy of sand and gravel. The cracking under this slurry loss was much wider than those experienced during the slurry loss of June 29, 1989 in panel 39 The guidewalls could be seen separating away from concreted panel Nos. 35 and 37. The guidewalls were also measured separating apart about an inch near panel 143. Total slurry lost in this event was estimated at 1,170 cy.

The contractor and Corps became concerned with the rate of progress; heightened by personnel, equipment, and project safety issues and the impending winter flood season, and entered into a joint effort to develop an acceptable solution to the slurry loss problem. Toward the end of June 1989 the contractor was requested to provide a technical proposal, cost proposal, and construction schedule for proceeding with cutoff wall construction in conjunction with a grouting program. The proposal was received a month later on July 20, 1989. A meeting was held with the contractor and geotechnical representatives from OCE, NPD, and NPS to discuss the merits and the contractor's proposal. The contractor's proposal was accepted with the exception that the embedment of the panels into bedrock would remain at 15 feet. The contractor had proposed to reduce the embedment to 5 ft. to reduce the excavation time, however, the possibility of not achieving adequate embedment outweighed any potential benefits.

Another meeting was held to address potential safety concerns and prepare questions for Dr. Ralph B. Peck, Geotechnical Consulting Engineer for the Corps. A task force meeting was held on August 17, 1989 with Dr. Peck and geotechnical representatives from OCE, NPD, and NPS. The only caveat to the proposed plan of action was that multiple lines of grout may be necessary to adequately pretreat the core. A single line of grout holes on either side of the cutoff wall alinement was considered possibly too narrow a band. It was also recommended that the closure area for the wall be predetermined so that the area of least risk to dam safety was completed last. Continuing work on the cutoff wall was favored as long as practical.

The last slurry loss occurred during excavation of panel 144 on August 18, 1989 at a depth of 151 feet. The slurry loss was controllable with thickened slurry and the addition of 9 bags of cement. About this time a crack began to propagate over the surface of the dam from panel 144 over a distance of 175 ft. along the guidewall. The crack opened to 1 3/4 inches over a two hour period. Excavation continued, however, and the panel was successfully concreted on August 24, 1989. This was the last panel completed before a temporary shutdown to implement the recompression grouting plan. The hydrofraise work did not restart until December 4, 1989.

INVESTIGATIVE DRILLING PROGRAM

LOCATION DRILLED PANEL # AND BITE	DATE DRILLED	DRILLED DEPTH (FT)	REMARKS	RUNNING TOTAL
17-C	6/7/89	73		73
17-A	6/8/89	138		211
19-C	6/9/89	172		383
19-A	6/10/89	158		541
35	6/12/89	152		693
37-A	6/13/89	146		839
37-C	6/13/89	132		971
33-C	6/14/89	166		1,137
33-A	6/15/89	190		1,327
31 (#2)	6/15/89	252	2 FT OFF RT SIDE OF PANEL 31	1,579
31 (#1)	6/16/89	360	2 FT OFF LT SIDE OF PANEL 31	1,939
29-C	6/19/89	385		2,324
23-A	6/20/89	280	LOST SLURRY @ 247 FT CAVING, PULLED RODS NOT GROUTED	2,604
23-B	6/22/89	303	AKA 23A-10	2,907
21/22	6/21/89	110	TOOK 54.4 CY GROUT LOCATED IN PANEL 22 AKA 23A+10	3,017
25 *	6/22/89	200	TOOK 85 CY GROUT	3,217
26	6/26/89	385	TOOK 60 CY GROUT	3,602
28 *	6/29/89	200	TOOK 35.6 CY GROUT	3,802
22 *	7/5/89	254	AKA 23A+8, NO TAKE	4,056
25 *	7/8/89	335	TOOK 4.8 CY	4,391
22 *	7/11/89	OPEN HOLE	TOOK 20.6 CY GROUT	4,391
28 *	7/12/89	360	TOOK 31.2 CY GROUT**	4,751
24	7/17/89	302		5,053
			291.6 CY GROUT TAKE IN	5,053 LF

* = Indicates these holes were re-drilled (reclaimed) in the exact same location.

** = This hole lost slurry 1-1/2 hours after a slurry loss occurred in panel 25 during excavation with the Hydrofraise.

TABLE VIII-1

SECTION IX

IN-SITU CORE MODIFICATION

SECTION IX - IN-SITU CORE MODIFICATION

A. Introduction. The large slurry loss problems encountered during the cutoff wall construction became more and more unmanageable from the standpoint of safety, constructability, schedule impact, and cost. The overriding safety concerns were:

1. Trench sidewall instability and potential cave-in at the work platform level endangering workers.

2. Continued propagation of cracks within the dam creating potential seepage paths during the impending flood storage season.

3. Trench sidewall instability and potential entrapment of the excavating machinery within the trench.

It was clear that slurry losses of this magnitude could not be addressed on a routine basis with the pre-planned slurry loss measures alone. A joint effort between the Corps of Engineers, Soletanche, Inc. and private consultants for both parties to establish a plan of action resulted in agreement on a major grouting program designed to pretreat the soil in the core.

B. Recompaction Grouting Procedures. The grouting plan consisted of two parts. The first part consisted of grouting under gravity feed to seal

existing cracks and fill existing voids. The second part consisted of pressure grouting selected zones within the core to prestress or recompact the core and prevent the occurrence of hydraulic fracture due to slurry pressure in the cutoff wall trench. The grout would be injected under pressure through sleeve pipes into the embankment core and would fill up existing cracks, create new hydraulic fractures and stress the soil surrounding the fractures and cracks; thereby compacting and increasing the soil stresses. The cracks and fractures would become filled with an impermeable grout.

Hydraulic fracturing of the core material is controlled by limiting the grouting pressure and quantities of grout placed during each grouting sequence. Repetition of this operation creates plane surfaces which intersect in multiple directions. This structure provides cohesion and resistance to further fracturing. Quantities of grout, flow rate, pressures, and number of repetitions of the grouting sequences are monitored closely to carefully control the grouting process. The gravity grouting was done by drilling 5 inch diameter holes on a 5-foot center to center spacing along the centerline of the cutoff wall alinement. The holes were rotary drilled with a Schramm T-685H and a Ingersoll Rand TH-60 truck mounted drills using a bentonite-cement drilling fluid/grout mix. The components per cubic yard were 460 pounds of cement, 55 pounds of bentonite and 180 gallons of water (about a 5 to 1 mix and 11 percent bentonite). When drill fluid losses occurred, sodium silicate was added into the bentonite-cement mixer to achieve a gel time ranging from 30 minutes to an hour. Sodium silicate reacts with the calcium in cement to produce a gelling

action. The mix without sodium silicate would provide an unconfined compressive strength on the order of 15 psi at 28 days. A high velocity mixer and PH15 grout pump were used in this operation. The PH15 is a high pressure, high capacity grout piston pump. It has an electrically driven 15 HP motor and can deliver a maximum of 45 gallons per minute at a maximum pressure of 1500 psi.

A total of 43 gravity grout holes were drilled taking 533 cubic yards of bentonite-cement grout and 808 gallons of sodium silicate. A detailed tabulation of hole depths and grout lost is provided on Table IX-1. The work was started on 8 August 1989 at hole No. 400 (approximate station 15+78.7) and completed on 30 August 1989 with the redrilling of hole locations 416 and 423.

The pressure grouting was accomplished through multiple sleeved port grout pipes placed in drilled holes and encased within a weak bentonite-cement mixture. The grout pipe consisted of 2" steel or plastic pipe with lateral perforations 3/16" diameter located every 2 feet. Each perforation was covered with a 4-foot long elastic sleeve to act as a non-return valve. Steel pipes were placed in the upper 150 ft., plastic pipes were placed in the lower part of the grout hole. This solution was aimed at avoiding interference of steel pipe with the future excavation of the Hydrofraise. This was a compromise solution since the recompaction grouting involved high pressures and potential soil displacement which plastic pipes are less capable to withstand. However, the

relatively good density of soil in the lower part of the dam (except locally) reduced the grouting requirements and permitted satisfactory use of the plastic grout pipes.

Sleeve grout pipes were manufactured in France for the secondary holes, but due to the early start of the first phase, it was necessary to improvise the construction of the primary grout pipe on the site. After tests, a sleeve made with shrink plastic tubing and electrical tape was selected and proved to be as effective as the regular grout pipe.

The pipes were installed along two lines on either side and parallel to the cutoff wall alinement. One line was installed 5 ft. upstream of the wall and the other 7 ft. downstream of the wall. The assymetry in the distance was due to the fact that the cutoff wall alinement was parallel to and 10 ft. upstream of the dam centerline. These holes were categorized further as primary grout holes spaced 12 ft. apart on center and secondary grout holes drilled between the primary grout hole locations. Therefore, in total, the grout pipes were installed on 6 ft. centers. The primary grout locations are all even numbered from 100 to 132 on the upstream line and from 200 to 232 on the downstream line. The secondary grout locations were all odd numbered from 101 to 131 on the upstream line and from 201 to 231 on the downstream line. The primary pipes were installed between August 31, 1989 and September 27, 1989. The secondaries were installed between October 16, 1989 and November 3, 1989.

The same rotary drills used for the gravity grouting phase were used to install the sleeve pipes. The holes were drilled with bentonite slurry to rock level with a tricone bit approximately 5 1/2" diameter. After withdrawing the drill string a 4" steel casing was lowered into the hole down to the rock contact. The grout pipe was placed into the hole under the protection of the casing. The grout pipe was then filled with water and plugged. Prior to withdrawing the casing, a bentonite-cement sealing mix was pumped in the annulus between the pipe and casing which displaced the bentonite slurry out of the annulus between the casing and soil. The sealing mix was pumped in as the casing was withdrawn.

The upstream row of the grout pipe installation holes and some of the gravity grout holes were drilled with the Schramm T-685H drill equipped with a drill parameter recorder marketed by Soletanche, Inc. as the Enpasol. This recorder was used to provide a qualitative density profile across the dam and identify probable high grout take zones.

In the fifties, the oil drilling industry attempted to correlate drilling parameters such as rate of drilling and the torque of the drilling rig with the characteristics of the drilled formation. Empirical formulas have been established by authors such as Teale, Somerton, etc..

The Enpasol recordings rely on the same basic principle. The Enpasol is a "black box" monitoring up to 8 drilling parameters. Every 5mm of depth, the

digitalized data are recorded on tape. Raw parameters are printed on paper. Computer processing of the parameters provides a clear picture of the stratigraphy of the drilled formations. On grouting projects, the Enpasol recordings display the stratigraphy of a formation at the very location where grout penetrates into soil. The following parameters were recorded:

- rate of drilling
- thrust on the drilling bit
- counterthrust
- torque
- pressure of the drilling fluid

The unit can also be outfitted to read rotation speed, reflected vibration and drill advance rate fully utilizing six pressure gauges, a speedometer (with cable and pulley), high pressure hose, electronic cable, and Compaq computer system. The systematic Enpasol recording of the upstream row of grout holes provided a quick, relative sense of the Mud Mountain Dam core density prior to initiating the recompression grouting program.

The raw data recorded by the Enpasol were interpreted by computer programs adjusted by Mr. Duchemin of Soletanche, Inc. to print out profiles of the drilled hole. The primary parameters used were torque, thrust and rate of drilling. The formulas used in the computer interpretation are given on figure IX-1. A typical profile is shown on figure IX-2. The darkest and broad

zones indicate low density areas. This profiling indicated that the upper zones (between 150 and 200 ft. deep) of the dam and areas close to the canyon walls (especially the left wall) were not as resistant to the drill (see figure IX-3. The worst zone was believed to be the deepest part of the left canyon wall. The Enpasol recorder was used on 10,103 LF of the drilling which represented approximately 31 percent of the total length of drilling for this grouting work.

The contractor utilized this information to establish a upstage grouting plan with injection volumes and maximum injection pressures preset for each 2-foot stage of the sleeve pipe. Generally the grout pumps were set to shut off when pressure exceeded 60 bar (880 psi). As refusals began to occur in the primary holes before the planned volume of grout had been injected, the maximum pressure was scaled back to 20 bar (300 psi) to limit the grout travel. The secondary holes were injected with a thicker grout to a maximum pressure of 60 bar. Generally 200 litres per 2 ft. stage was planned in the upper 3/4 of the hole and 400 litres per 2 ft. stage in the lowest 1/4 of the hole. The range of volumes selected were from 150 to 200 litres per stage in the upper reaches and 200 to 1000 litres per stage in the lower reaches.

Two basic stable grout mixes designed by the contractor were used. The primary holes were injected with a thinner grout to fill any existing voids and cracks. The grout mix consisted of 470 pounds of cement, 83 pounds of bentonite and 178 gallons of water per cubic yard (about a 4.75 to 1 mix with

17 percent bentonite). The secondary holes were grouted using a mix of 940 pounds of cement, 100 pounds of bentonite and 167 gallons of water per cubic yard (about a 2.25 to 1 mix with 10 percent bentonite).

A limited amount of sodium silicate was used in both the primary and secondary grout mixes. Typically 3 to 8 gallons per cubic yard of grout were added to provide rigidity and setting times on the order of 30 minutes to 1 hour. In the upper 150 ft. of the dam short set times were not sufficient to prevent the extrusion of grout at the surface. To prevent such leakage, silicate and grout were pumped simultaneously down the hole through a specially made manifold. This mix achieved a flash setting time of 2 minutes.

On the average, grout was injected at a rate of 4 gallons per minute in the primary holes and at 2 gallons per minute in the secondary holes. The rate was reduced for the secondary holes due to the thicker grout mix used and the faster pressure rise noted as the grouting program progressed. The intent of the program was not to achieve maximum penetration or permeation, but to induce maximum stressing and compression of the soil bounding the future cutoff wall excavation.

The grout station was a containerized unit shipped from France with 4 double (total of 8 pumps) high pressure pumps (PH 2x5) which could deliver a maximum of 8 gallons per minute and inject grout at a maximum pressure of 1200 psi. The pumps are powered by a 5 KVA electric motor. Each pump was hooked up

to a Kent flow and pressure meter with circular graph recorder and digital display. The pumps could be preset for injection of specific grout quantities and/or a maximum pressure build-up. Gradual build-up of pressure on the grapn recorder gave indication that the soil was being properly stressed. A flat pressure indicated voids, cracks or soft soil zones.

Behind the containerized pump and recorder station were two 3 cy capacity grout mixers and an adjacent 50 ton cement silo. One grout mixer was a high velocity type for cement bentonite grout. The other was a low velocity mixer for sodium silicate treated grouts.

Large mobile spools held 400 ft. of high pressure grout hose and a high pressure water line for inflation of the double packer at the end. The spool was 4 ft. in diameter with manual pumps for packer inflation and pressure gauges for both packer and grout pressure. The spool was called a "Joseph" after Mr. Joseph Dietsch who designed the unit and also managed the site set-up on this and other Hydrofraise cutoff wall contracts. A schematic diagram of the grouting setup is shown on figure IX-4.

The sleeve pipes and double packer system allowed grouting to be done in several passes at various stages from the same pipe. Proper water jet washing of the pipe after each application allowed the same stage to be grouted more than once. Up to five passes were made at the same stage in some pipes. This allowed a gradual build up of resistance and stressing of the soil. The intent

was to recompact the zone of soil 12 ft. upstream and downstream of the cutoff wall or about 52,135 cy of the dam core.

C. Recompaction Grouting Results. A detailed listing of grout injection plans, deviations from the plan and total grout and silicate injected in each primary and secondary hole is given in tables IX-2 to IX-14. Above and beyond the 533 cubic yards of grout and 808 gallons of sodium silicate placed during the gravity grouting phase, the recompaction grouting program injected another 4,550 cy of grout and 17,785 gallons of sodium silicate. Prior to both these programs an experimental investigative drilling and grouting program completed between June 7 and July 17, 1989, injected 292 cubic yards of grout under gravity feed. Therefore, in total, about 5,375 cy of grout were injected to treat 52,135 cy of the core or about 10 percent by volume. According to the contractor's experience, a range of 3 to 10 percent by volume of treated soil was expected. Mud Mountain Dam therefore took grout on the high side of this expected range. A summary of grout quantities is given in table IX-15.

The grouting program was completed on December 1, 1989 and cutoff wall excavation resumed on December 4, 1989. Cutoff wall excavation continued on through April 10, 1990 and was completed without any further slurry losses. The recompaction grouting program can only be deemed a complete success.

D. Reactivated Dam Settlement. Settlement on top of the dam had been monitored while the Hydrofraise excavation was ongoing and prior to the

grouting program. During this time the guidewalls on each side of the cutoff wall began to settle about 1" over a broad downward curve across the deep section of the dam. This could be attributed strictly to structural settlement of the 4 foot deep 18 inch thick concrete guide wall segments. Later during a two week period while recompaction grouting of the core, the settlement accelerated and dropped the downstream guidewall another 4 inches. The ground surface on top of the dam immediately adjacent to the downstream guidewall also dropped at a relatively abrupt boundary. A maximum settlement of about 1/2-foot occurred between August 30, 1989 and December 14, 1989. The upstream guidewall settled only half as much. The peak settlement occurred at about station 15+00. This was almost directly over the vertical drop and step on the bottom left canyon wall where the Enpasol recording and grout takes indicated voids or soft soil conditions.

In addition to these vertical settlements along the two guidewalls, the horizontal as well as vertical movements of survey monuments along each edge of the top of dam were measured. These monuments indicated that the outside shoulders of the top of dam were moving outward (see table IX-16). The upstream line moved out further upstream and the downstream line moved out further downstream. The vertical measurements indicated that the upstream line lifted up about 1 inch at station 14+50, while settling about 1.4 inches at station 16+00 and 13+50. The downstream line had a similar shaped curve but no actual uplift. The survey data is included in the appendix to this report.

The various movements that occurred during the recompaction grouting program could be explained by deep and near surface settlement mechanisms and surface heave caused by the grouting. The settlement of the guidewalls was not expected and not consistent with surrounding ground surface movements on top of the dam. Since the guidewalls were in between the two pressure grout lines, the maximum heaving effect was expected to occur along the guidewall/cutoff wall alinement. Heave did occur upstream and downstream of the upstream and downstream pressure grout lines, however, the guidewalls and immediately adjacent soil continued to settle as a downdropping block about 10 ft. in width and encompassing the guidewall/cutoff wall alinement. Heaving on top of the dam was a readily observable condition (see figures IX-5,6,7, and 8) while the outward movement of the shoulder monuments on top of the dam were detected by surveys.

The accelerated 1/2-foot settlement of the downdropping block over a two week period of grouting can be explained in the following manner:

1. The heave induced at the top of the dam produced a radial pressure front pushing the shoulders out and the center up. In the process, tensile stresses had to be induced to accommodate such a heave geometry which in turn created sub-vertical longitudinal cracks. Two of these cracks defined the outside boundaries of the downdropping block.

2. The block failed to heave because of near surface softening of the core

material to a somewhat semi-viscous state due to the injection under gravity feed of weak cement-bentonite grout along the cutoff wall alinement prior to the recompression grouting and the relatively slow setting time of the near surface recompaction grout.

The near surface grouting was difficult due to rapid escape of grout to the surface through sub-vertical cracking. It was not until later that sodium silicate and grout were injected simultaneously down a specially made twin manifold to cause a flash set-up in the grout.

In conjunction with the near surface settlement mechanism and heaving, there is also a deep surface settlement mechanism to explain continued long term settlement of the top of dam and downdropped block. Surveys of the dam slopes indicate accelerated settlement (2 inches in 3 months) of these surfaces when compared to historical data (12 inches in 35 years, 1950 to 1985). This was evidence that deep core areas had begun to settle again as the injection of grout at depth destroyed soil structures (arches, etc. and may have displaced or allowed slurry trapped within the core from previously experienced hydro-fracture to escape). This was viewed as a positive sign that densification effects were being produced.

In addition to normal considerations of arching across narrow steep canyons and transverse arching against relatively stiff rock shells, a great deal of evidence points to a site specific problem with the bottom left canyon wall.

Piezometer readings gave indication during high pools that a major open zone existed in the canyon walls and especially on the left side. The Enpasol data indicated loose soil or voids along the left canyon wall. Loss of fines from the core material occurred near the base of the dam and notably near the left canyon wall due to piping, flushing during reservoir drawdown, open rock joints, and/or chemical action of subsurface water on dental concrete. The peak settlement along the guidewall occurred almost directly over the left canyon wall and the grout takes were high in this area. Because of these corroborating facts, a low pressure zone may have allowed settlement of the soil column above and established arching similar to a trap door analogy under a silo of soil. The arching established before cutoff construction was then destroyed by the cutoff wall excavations and grout injections contributing to the overall and on-going settlement.

These settlement and heave mechanisms were discussed along with overall integrity considerations in three memorandums which are enclosed in the appendix to this report. Of particular concern for those involved in the long term monitoring of the cutoff wall are two things which should be kept in mind. First, flushing action on the upstream core material during drawdown will be an on-going concern not mitigated by the cutoff wall construction. Continued loss of support on the upstream face of the wall may make the downstream soil pressures more a loading concern than hydrostatic reservoir water pressures. Bending upstream down low in the wall combined with downdrag on the wall due to settlement may cause eventual cracking of the cutoff wall. The inclinometer

readings should give advance notice of such a condition occurring. Second, piping action due to seepage through rock joints may not have been fully eliminated. The pre-cutoff piezometric data had indicated a major open zone along the left canyon wall and presumed to be primarily a core failure phenomena. It may be a sink in the rock wall which may exist and continue to present a long term seepage problem. Failure is certainly not imminent and future piezometric readings during pool rise and fall should define the magnitude of any problem. A limited bedrock grouting program would probably be the quickest solution should a problem be discovered.

ENPASOL RECORDINGS for PRESSURE GROUTING HOLES

Drill hole number	Recorded length ft	Drilled length ft
101	134	159
103	123	173
105	178	178
107		
109	258	258
111	271	271
113	297	297
115	315	315
117	356	356
119	332	384
121	385	385
TOTALS	2649	2776

Drill hole number	Recorded length ft	Drilled length ft
123		
125	383	383
127	350	375
129	243	243
131	189	189
223	303	303
225	376	376
229	179	204
231	183	183
TOTALS	3823	4622

The ENPASOL has recorded the following data :

P0 Drilling depth (ft)
P1 Drilling fluid pressure (PSI)
P2 Torque(PSI)
P3 Feed pressure(PSI)
P4 Speed rate of drilling (ft/h)
P6 Retention pressure (psi)
P8 Time (0.0164ft of drilling)

Combined parameters :

A = Actual THRUST on drillbit (kips) = (P3/S1 − P6/S2)/1000

rem.: S1 = section of feed hyd. jack = 7.07si
S2 = section of retention jack = 3.93si

T = Relative corrected TORQUE (with P0) (PSI) = (P2 − 2*P0)/100

R = Rig EFFECT = sqrt(A) * T / 30

Relative LOOSENESS of the soil = (P4 / (R+0.01))^2 − 0.5

rem.: this formula has been used to illustrate on the graph the contrast between the different kinds of soils.

General remark : All the parameters (outputs) have been smoothed to 0.5 ft

FIGURE IX-1

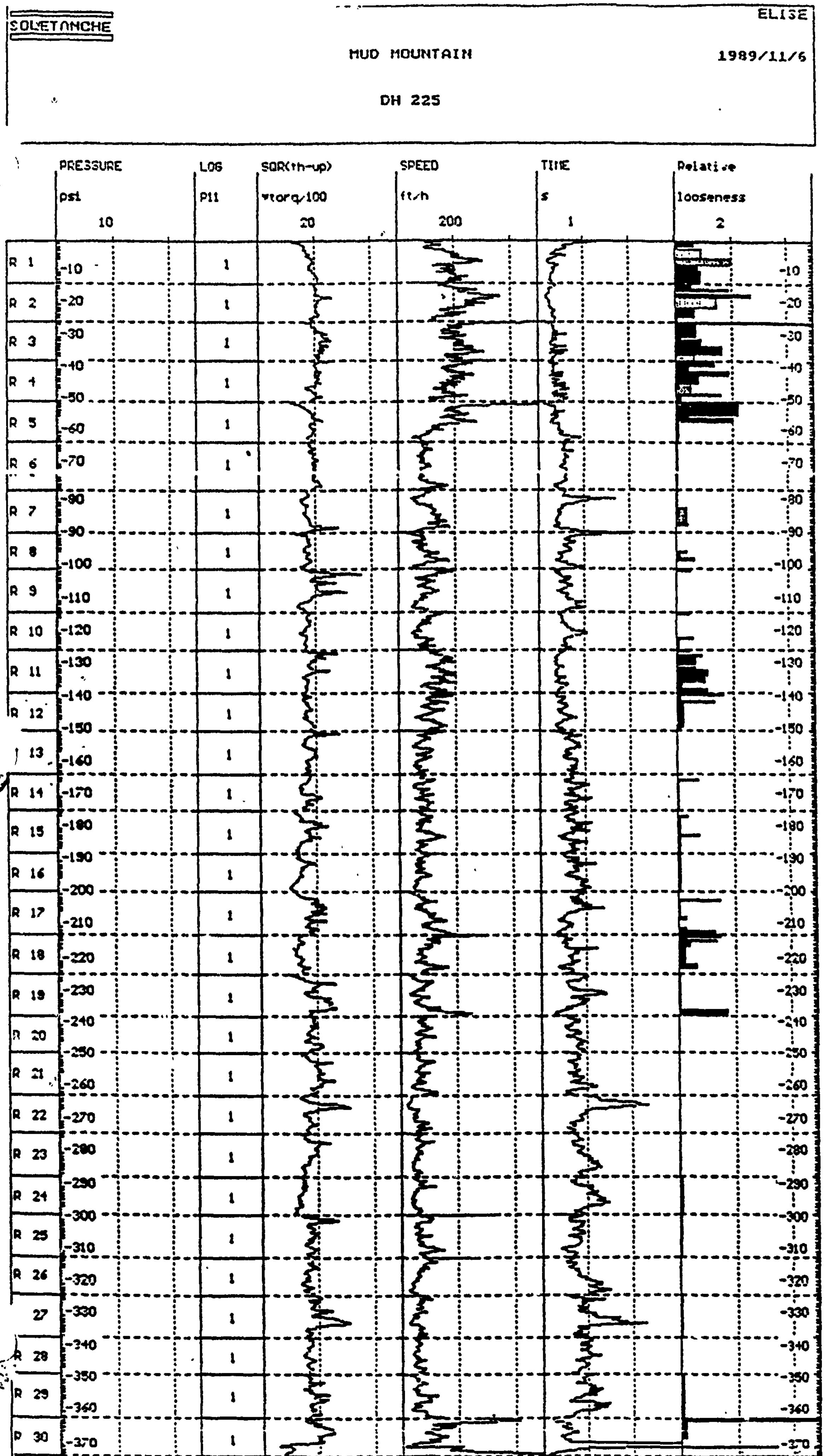

SOLETANCHE
ELISE
MUD MOUNTAIN
1989/11/6
DH 225
PRESSURE
psi
10
LOG
P11
SQR(th-up)
*torq/100
20
SPEED
ft/h
200
TIME
s
1
Relative
looseness
2
R 1
R 2
R 3
R 4
R 5
R 6
R 7
R 8
R 9
R 10
R 11
R 12
13
R 14
R 15
R 16
R 17
R 18
R 19
R 20
R 21
R 22
R 23
R 24
R 25
R 26
27
R 28
R 29
P 30
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
-150
-160
-170
-180
-190
-200
-210
-220
-230
-240
-250
-260
-270
-280
-290
-300
-310
-320
-330
-340
-350
-360
-370

FIGURE IX-2

ENPASOL RECORDINGS

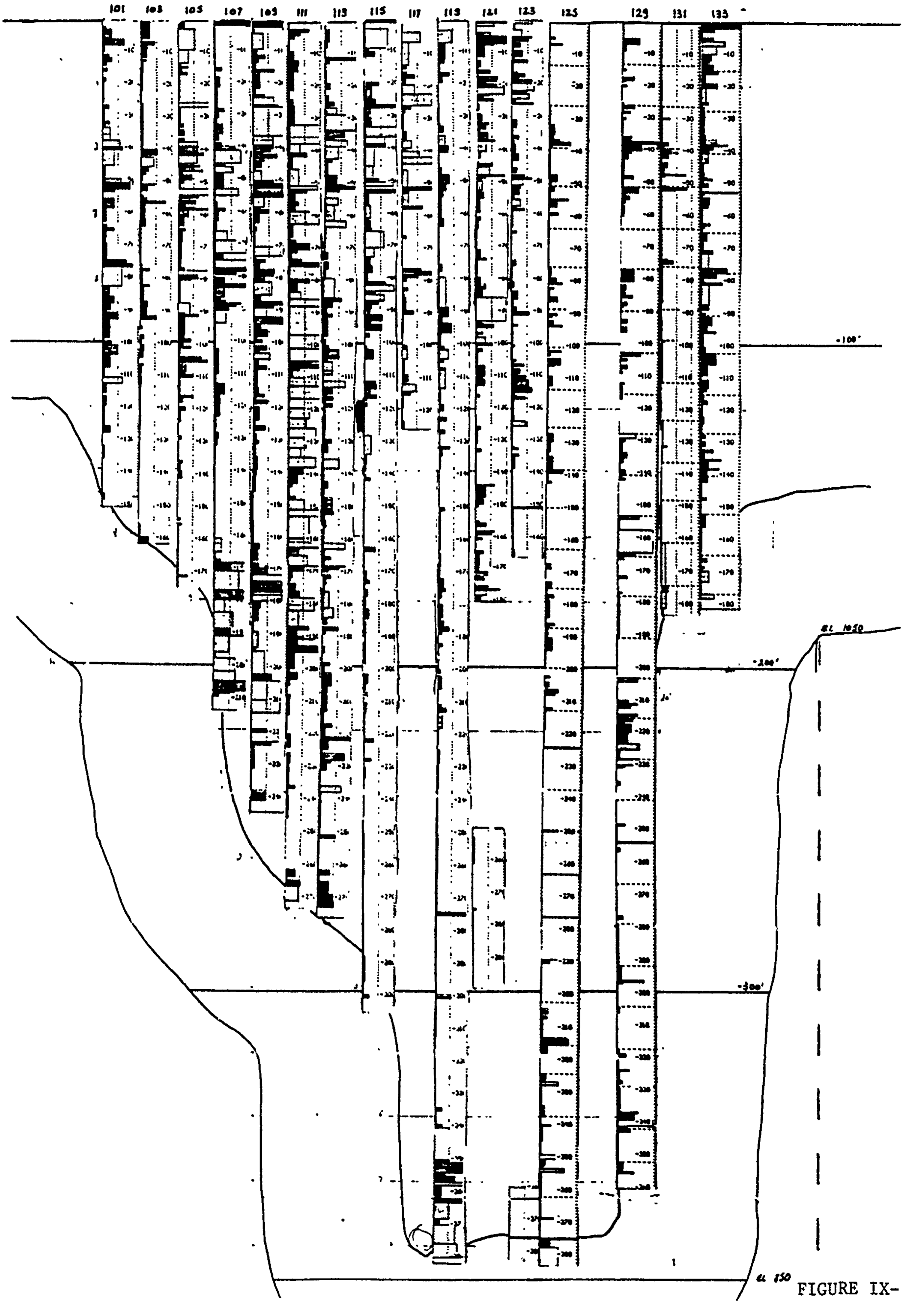
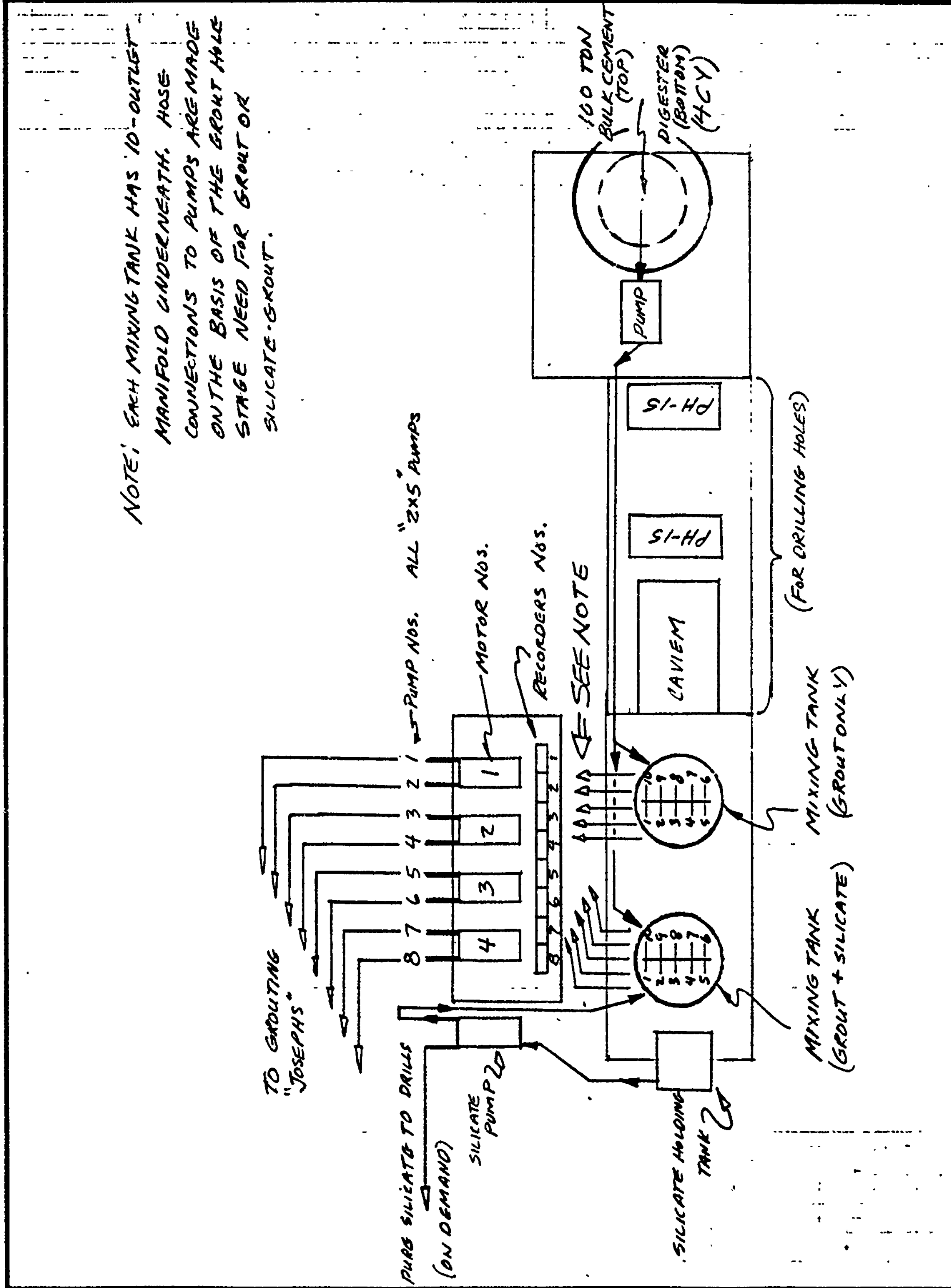


FIGURE IX-

U.S. ARMY CORPS OF ENGINEERS   OFFICE SYMBOL:

PROJECT: Mud Mtn Dam Seepage Control Project	COMPUTED BY: M. Satter	DATE: 9/20/89 SHT. OF
SUBJECT: Compaction Grouting Physical Plant Layout	CHECKED BY:	PART:



NPD FORM 7 (REVISED) JUNE 86   10 GRID

FIGURE IX-4

Mud Mountain Dam Deck Elevations

X-Section at 14+00 Looking SE

Surveys by MMD Resident Office
Controlled by reference to
Horton-Dennis Surveys of 10/13, 11/13/89
Closed to 0.014ft and corrected 12/4 survey
Closed to 0.033ft but not corrected 11/17 survey

12/7/89

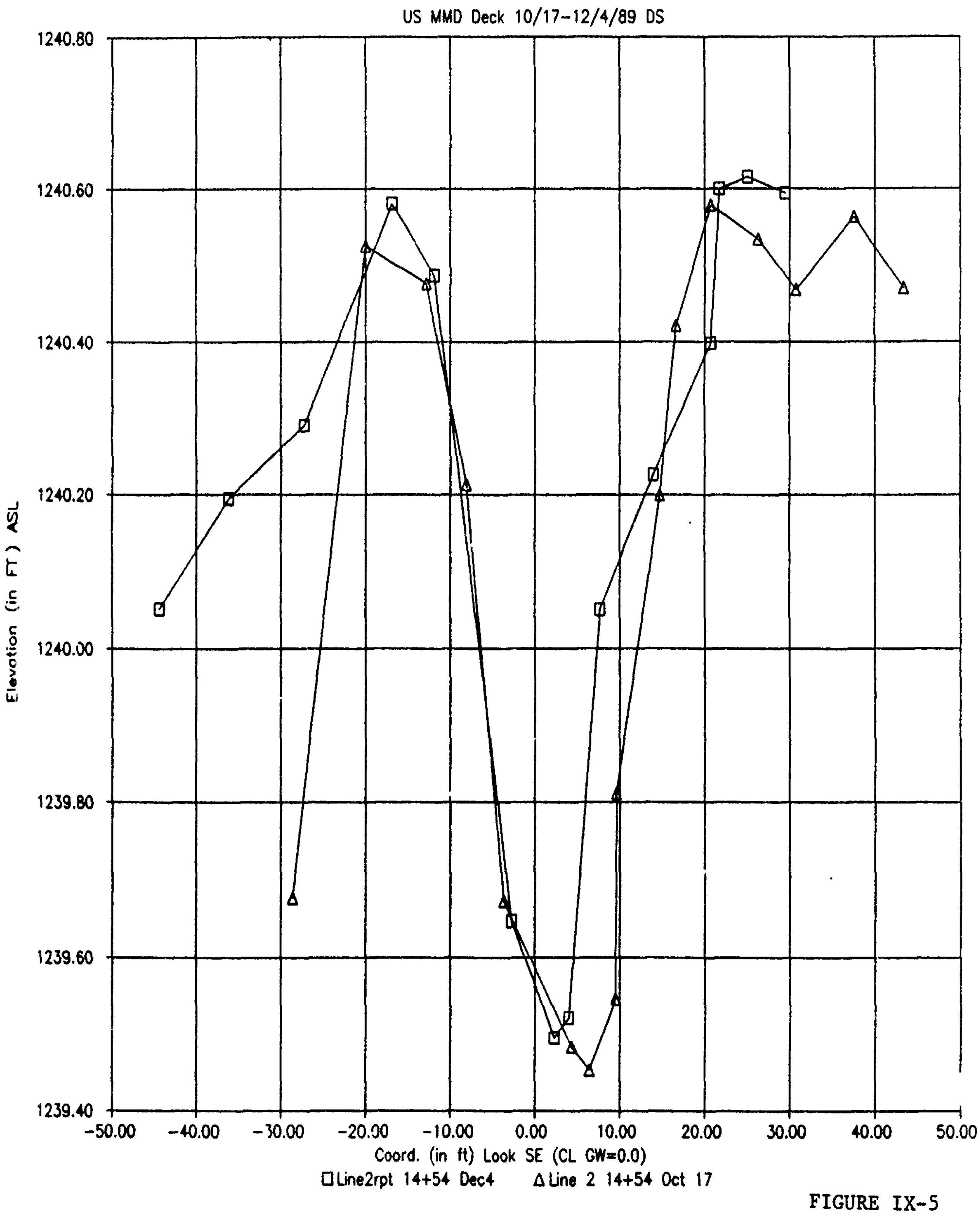


FIGURE IX-5

smj'89

Mud Mountain Dam Deck Elevations

X-Sections at 14+00 and 14+54

Looking SE

Surveys by MMD Resident Office
Controlled by reference to
Horton-Dennis Surveys of 10/13, 11/13/89
Closed to .033 ft and .017 ft., respectively
10/17 uncorrected, 12/4 corrected

12/7/89

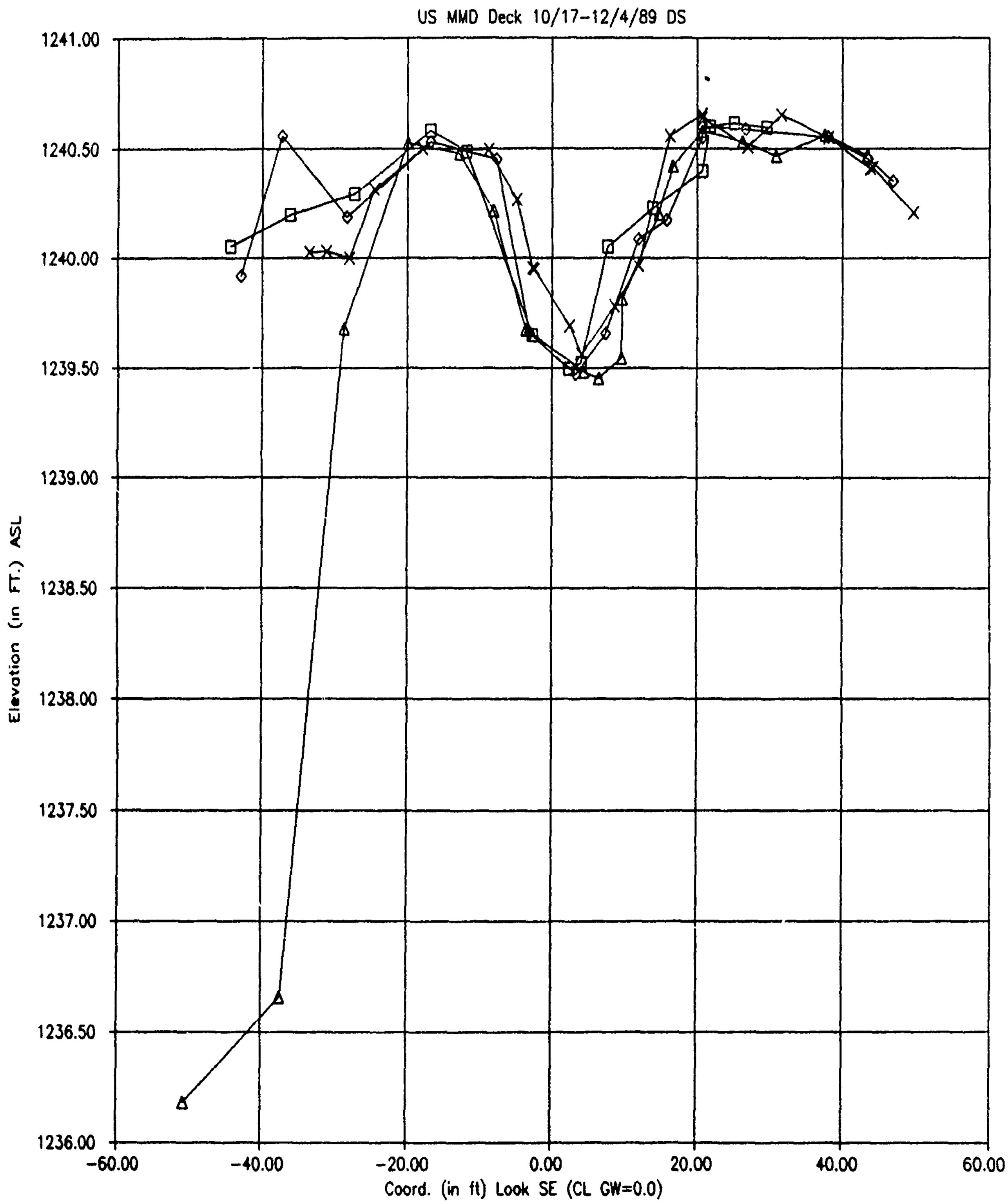


FIGURE IX-6

smj'39

Mud Mountain Dam Deck Elevations

X-Section at 14+54 Looking SE

Surveys by MMD Resident Office
Controlled by reference to
Horton-Dennis Surveys of 10/13, 11/13/89
Closed to .033 and .017 ft., respectively
10/17 uncorrected and 12/4 corrected

12/7/89

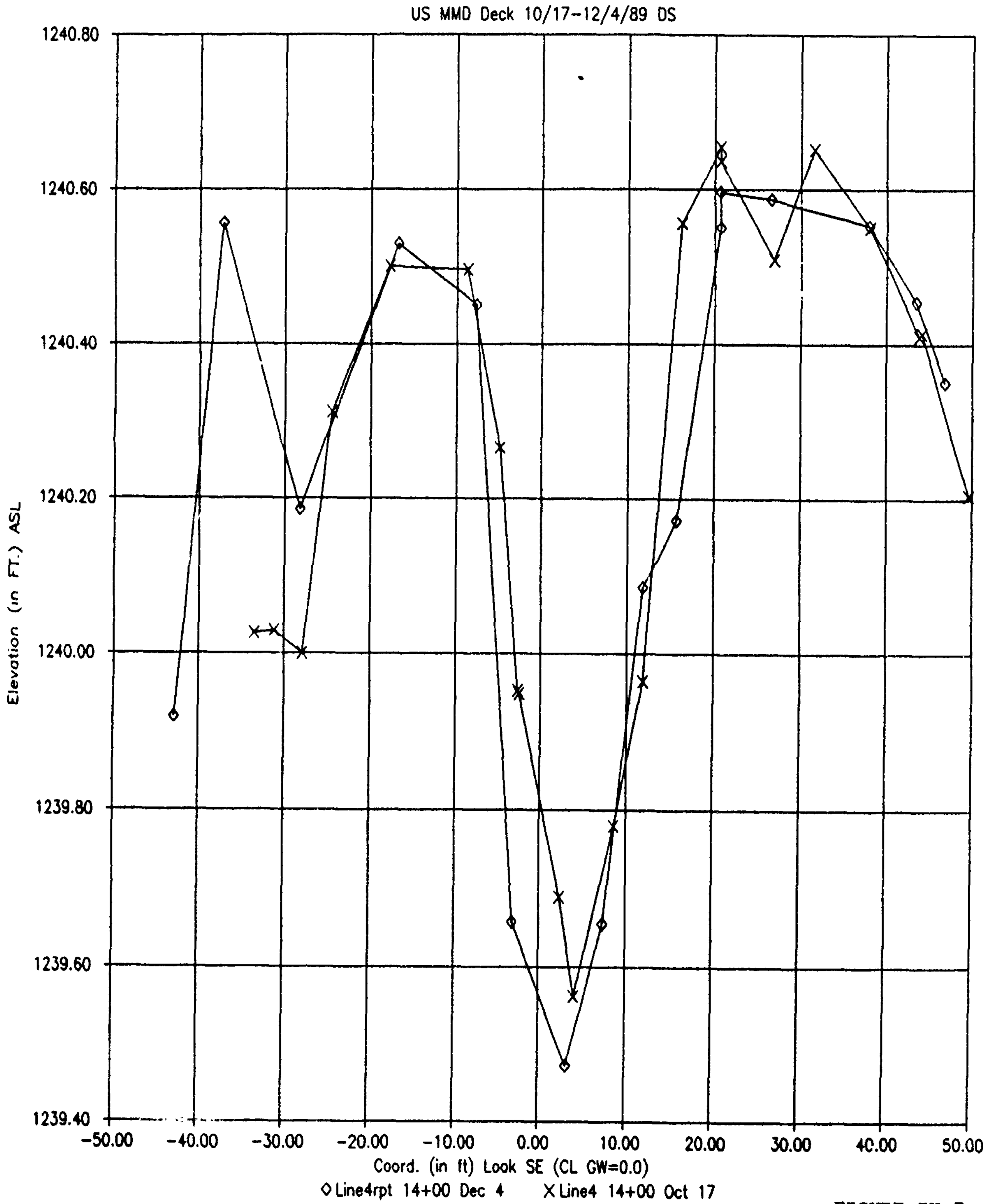


FIGURE IX-7

sj'89

Mud Mountain Dam Deck Elevations

X-Sections from 14+00 to 15+20 Looking SE

Surveys by MMD Resident Office
Controlled by reference to
Horton-Dennis Survey of 10/13/89
Closed to 0.033ft avg. bust
Uncorrected data lines 2,3,4
Corrected data for line 1

Nov. 8, 1989

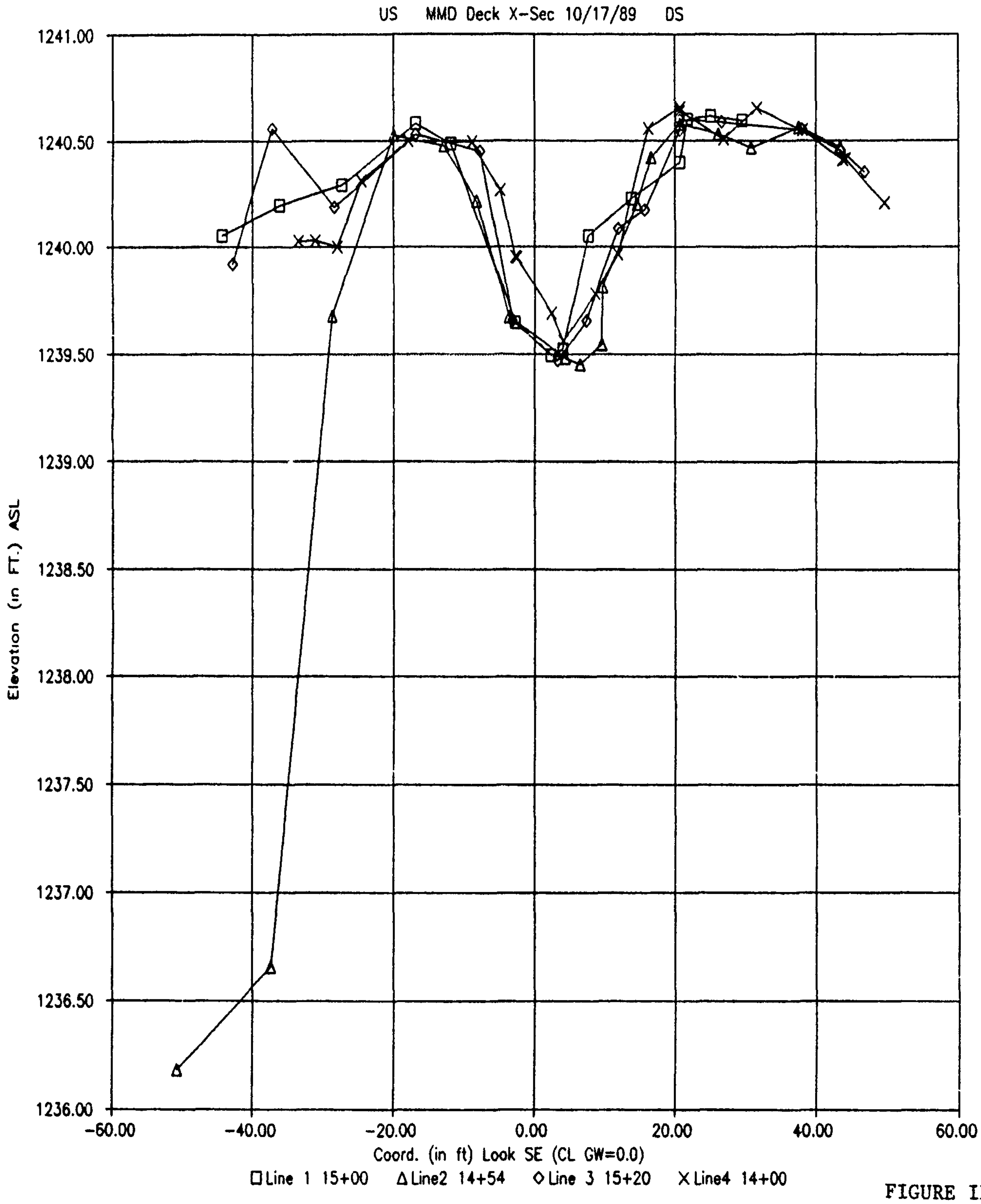
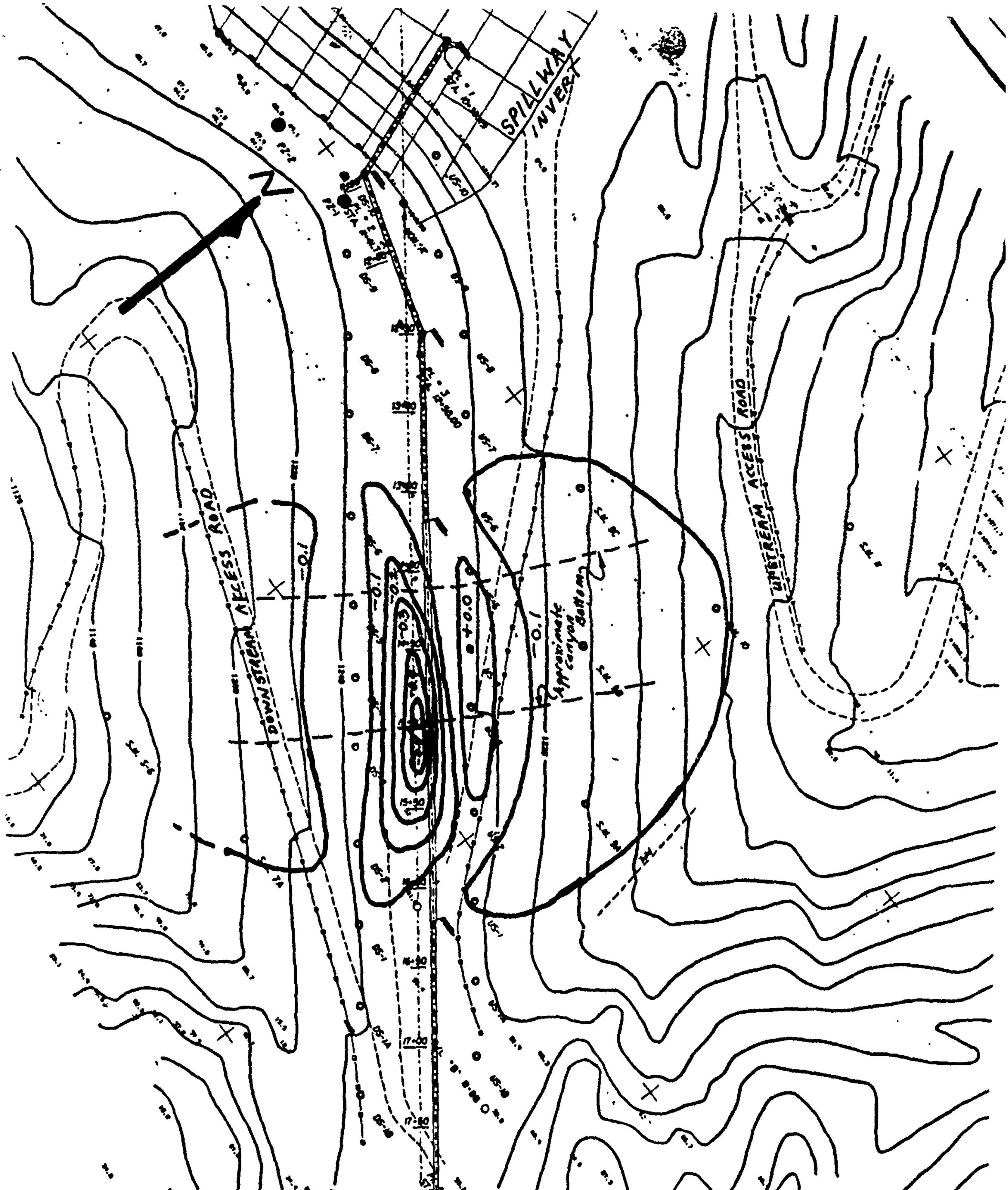


FIGURE IX-8
SMJ'89



NOTE: Settlement contours in feet.

FIGURE IX-9

CASE 17 GRAVITY GROUTING

/BORING NUMBER/	/DRILL/	/DATE/	/TOR/	/DEPTH/	/GROUT/	/GROUT DEPTH/	/TOTAL GROUT/	/NEATLINE GROUT/	/ % OF NEATLINE/	/GROUT LOST/	/ GALS OF SILICATE/	/REMARKS/
No. 400	SCHRAMM	08-Aug-89	160.00	166.00	3.30	166.00	10.50	1.06	988.33	9.44		
No. 401	SCHRAMM	16-Aug-89	162.00	163.00	3.50	163.00	3.50	1.04	335.51	2.46		
No. 402	SCHRAMM	09-Aug-89	167.00	173.00	2.70	173.00	6.30	1.11	569.00	5.19		
No. 403	TH60	18-Aug-89	171.50	172.00	1.50	172.00	1.50	1.10	136.26	0.40		
No. 404	SCHRAMM	28-Aug-89	174.00	175.00	4.80	175.00	4.80	1.12	428.57	3.68		
No. 405	SCHRAMM	16-Aug-89	177.00	177.00	3.70	177.00	3.70	1.13	326.62	2.57		
No. 406	SCHRAMM	10-Aug-89	202.00	202.00	23.00	55.00	26.00	1.29	2,011.14	24.71	50.00	
No. 407	SCHRAMM	29-Aug-89	244.00	244.00	4.40	244.00	4.40	1.56	281.76	2.84		Shared 4.8 cy with #433.
No. 408	SCHRAMM	16-Aug-89	238.00	238.00	1.20	238.00	1.20	1.52	78.78	0.00		
No. 409	SCHRAMM	24-Aug-89	264.00	264.00	3.20	264.00	3.20	1.69	189.39	1.51		
No. 410	TH60	21-Aug-89	256.00	256.00	4.70	256.00	4.70	1.64	286.87	3.06		
No. 411	SCHRAMM	24-Aug-89	277.00	277.00	2.00	277.00	2.00	1.77	112.82	0.23		
No. 412	SCHRAMM	17-Aug-89	285.50	286.00	38.40	110.00	42.20	1.83	2,305.51	40.37	37.00	
No. 413	TH60	22-Aug-89	328.00	328.00	1.50	328.00	1.50	2.10	71.46	0.00		Drill deflected at 200 ft.
No. 414	SCHRAMM	10-Aug-89	293.00	293.00	54.00	255.00	60.50	1.88	3,226.32	58.62	165.00	
No. 415	SCHRAMM	25-Aug-89	N/A	350.00	4.00	350.00	4.00	2.24	178.57	1.76		Hit steel at 350 ft.
No. 416A	SCHRAMM	17-Aug-89	385.00	386.00	66.00	369.00	79.20	2.47	3,205.96	76.73	110.00	Drill deflected at 290 ft.
No. 416B	TH60	30-Aug-89	382.00	382.00	3.60	382.00	3.60	2.44	147.25	1.16		Second time +2' sta.
No. 417	SCHRAMM	24-Aug-89	380.00	381.00	3.60	381.00	3.60	2.44	147.64	1.16		Drill deflected at 300 ft.
No. 418	SCHRAMM	21-Aug-89	345.00	345.00	34.80	145.00	34.80	2.21	1,576.09	32.59	69.00	
No. 419	TH60	23-Aug-89	358.00	358.00	2.40	358.00	2.40	2.29	104.75	0.11		
No. 420	SCHRAMM	11-Aug-89	N/A	360.00	97.00	255.00	117.10	2.30	5,082.47	114.80	15.00	Terminated early.
No. 420	SCHRAMM	14-Aug-89			14.40	360.00	N/A	N/A	N/A	N/A	278.00	Losses after weekend.
No. 421	SCHRAMM	29-Aug-89	379.00	380.00	5.30	380.00	5.30	2.43	217.93	2.87		
No. 422	SCHRAMM	18-Aug-89	380.00	381.00	6.00	381.00	6.00	2.44	246.06	3.56	69.00	
No. 423A	SCHRAMM	28-Aug-89	N/A	275.00	4.98	275.00	4.98	1.76	282.95	3.22		Plugged bit, terminated.
No. 423B	SCHRAMM	30-Aug-89	378.00	379.00	3.60	379.00	3.60	2.43	148.42	1.17		Second time same sta.
No. 424	SCHRAMM	15-Aug-89	381.00	381.00	7.80	381.00	7.80	2.44	319.88	5.36		
No. 425	SCHRAMM	25-Aug-89	379.00	379.00	4.50	379.00	4.50	2.43	185.52	2.07		
No. 426	TH60	24-Aug-89	381.00	381.00	2.60	381.00	2.60	2.44	106.63	0.16		
No. 427	SCHRAMM	28-Aug-89	381.00	381.00	3.00	381.00	3.00	2.44	123.03	0.56		
No. 428	SCHRAMM	22-Aug-89	377.00	377.00	3.90	377.00	3.90	2.41	161.64	1.49		
No. 429	TH60	25-Aug-89	380.00	380.00	4.00	380.00	4.00	2.43	164.47	1.57		
No. 430	SCHRAMM	24-Aug-89	367.00	369.00	2.40	369.00	2.40	2.36	101.63	0.04		
No. 431	TH60	28-Aug-89	373.00	375.00	13.30	375.00	13.30	2.40	554.17	10.90		
No. 432	SCHRAMM	23-Aug-89	N/A	307.00	1.70	307.00	1.70	1.96	86.52	0.00		Hit steel at 307 ft.
No. 433	TH60	29-Aug-89	298.00	298.00	5.40	298.00	5.40	1.91	283.14	3.49		Shared 4.8 cy with #407.
No. 434	TH60	28-Aug-89	333.00	333.00	3.60	333.00	3.60	2.13	168.92	1.47		Deviated off cyn wall.
No. 435	TH60	29-Aug-89	191.00	192.00	3.60	192.00	3.60	1.23	292.97	2.37		
No. 436	SCHRAMM	15-Aug-89	191.00	191.00	17.50	60.00	21.60	1.22	1,767.02	20.38	15.00	
No. 437	TH60	29-Aug-89	N/A	157.00	2.10	157.00	2.10	1.00	209.00	1.10		Refusal, terminated.
No. 438	SCHRAMM	16-Aug-89	171.00	172.00	9.40	172.00	9.40	1.10	853.92	8.30		
No. 439	SCHRAMM	29-Aug-89	167.00	168.00	2.20	168.00	2.20	1.08	204.61	1.12		
No. 440	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A		Could not be drilled, grout station in way.
No. 441	SCHRAMM	15-Aug-89	154.00	154.00	1.00	154.00	1.20	0.99	121.75	0.21		
			TOTAL FT:	12,386.00		TOTAL CY GROUT:	532.88	79.27	CY GROUT LOST:	454.[illegible]	808.00	US GALS SILICATE

TABLE IX-1

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-88-C-0047 ******DATA DATE: 11/02/89 *********

PRIMARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****FIRST PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT / REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
100	9/09/89	9/21/89	150	148 TO	142	400	140 TO	12	200	130'/160L,130'/160L,122'/50L	224.95	282.52	20.927	134
102	9/18/89	9/21/89	162	160 TO	154	400	152 TO	16	200	16'/180L	254.27	288.87	21.398	142
104	9/20/89	9/21/89	172	170 TO	154	400	162 TO	14	200	130'170L,14'/140L	0.00	583.05	43.189	154
106	9/20/89	9/21/89	210	208 TO	202	400	200 TO	12	200	164'/140L,	0.00	725.36	53.731	194
108	9/21/89	9/23/89	242	240 TO	172	400	170 TO	30	200	NONE	0.00	995.88	73.769	208
110	9/21/89	9/25/89	274	272 TO	262	400	260 TO	24	200	24'/150L	0.00	923.48	68.406	246
112	9/25/89	9/26/89	276	274 TO	266	400	264 TO	10	200	NONE	0.00	974.69	72.199	262
114	9/25/89	9/27/89	306	304 TO	24	200	TO			NONE	0.00	995.88	73.769	280
116	9/27/89	9/29/89	316	314 TO	308	400	306 TO	12	200	120'/80L,118'/10L	0.00	1101.82	53.989	300
118				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE.				0
120	9/27/89	10/02/89	382	382 TO	374	400	372 TO	32	200	262-274'/400L,32'/30L	0.00	1307.70	63.981	348
122	9/26/89	9/28/89	380	378 TO	370	400	368 TO	58	200	58'/100	0.00	1171.74	72.249	318
124	9/27/89	9/29/89	380	378 TO	372	400	370 TO	24	200	NONE	0.00	1235.46	67.510	352
126	9/29/89	10/0?/89	362	380 TO	362	400	360 TO	10	200	NONE	0.00	1384.34	67.730	368
128	10/02/89	10/04/89	358	356 TO	348	400	346 TO	10	200	116'/100L,114'/40L	0.00	1252.62	61.579	344
130	10/02/89	10/03/89	198	196 TO	178	400	176 TO	58	200	58'/20L	0.00	559.68	27.334	136
132	10/02/89	10/03/89	180	178 TO	166	400	164 TO	42	200	58'/70L,42'/0L	0.00	525.13	25.692	134
134				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	479.22	14363.22	-------	3920 LF OF HOLE GROUTED
										TOTAL GROUT U/S LINE:	14842.44 CF		867.371 USG SILICATE	

PRIMARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LINE ****FIRST PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT / REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
200	9/15/89	9/19/89	160	156 TO	20	200	TO			NONE	346.68	169.51	12.556	138
202	9/16/89	9/19/89	174	172 TO	166	400	164 TO	50	200	48'/160L	240.14	220.36	16.323	120
204	9/19/89	9/21/89	212	210 TO	204	400	202 TO	24	200	24'/70L	0.00	687.58	50.932	184
206	9/20/89	9/21/89	240	238 TO	232	400	230 TO	28	200	172'/130L,96'/90L,28'/140L	0.00	769.15	56.974	208
208	9/21/89	9/25/89	254	252 TO	170	400	168 TO	36	200	36'/140L	0.00	1064.39	78.843	214
210	9/21/89	9/25/89	282	280 TO	272	400	270 TO	46	200	NONE	0.00	866.74	64.351	232
212	9/25/89	9/27/89	308	306 TO	298	400	296 TO	34	200	34'/100L	0.00	999.41	74.030	270
214	9/25/89	9/27/89	330	328 TO	320	400	318 TO	34	200	NONE	0.00	1080.63	80.047	292
216	9/25/89	9/27/89	380	378 TO	360	400	358 TO	30	200	NONE	0.00	1306.64	90.572	346
218	9/27/89	9/29/89	380	378 TO	360	400	358 TO	26	200	NONE	0.00	1320.77	80.606	350
220	9/25/89	9/27/89	384	382 TO	374	400	372 TO	32	200	NONE	0.00	1278.39	87.058	348
222	9/26/89	9/29/89	384	382 TO	370	400	368 TO	24	200	NONE	0.00	1320.77	72.258	356
224	10/02/89	10/04/89	366	364 TO	356	400	354 TO	22	200	22'/20L	0.00	1300.99	63.652	340
226	9/27/89	9/29/89	366	364 TO	358	400	356 TO	54	200	68'/30L	0.00	1124.07	58.726	308
228	9/29/89	10/03/89	294	292 TO	284	400	282 TO	38	200	200'/100L,132'/0L REFUSAL	0.00	928.78	45.441	252
230	10/02/89	10/03/89	194	192 TO	174	400	172 TO	28	200	NONE	0.00	646.26	31.619	162
232	10/02/89	10/03/89	172	170 TO	162	400	160 TO	34	200	NONE	0.00	522.66	25.572	134
234				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	536.22	15609.10	-------	4254 LF OF GROUTED
										TOTAL GROUT D/S LINE:	16195.32 CF		989.560 USG SILICATE	

TABLE IX-2

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-88-C-0047 ******DATA DATE: 11/03/89 *********

PRIMARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****SECOND PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE /BOTTOM/ /TOP/	/LITRES PER STG/	REACH TWO /BOTTOM/ /TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
100	9/28/89	9/29/89	150	148 TO 142	400	140 TO 18	200	146'/10L,144'/10L,136-126';120';108'/0L,	0.00	373.28	18.263	128
								REFUSAL,32'/110L				0
102	9/28/89	9/29/89	162	160 TO 154	400	152 TO 36	200	154',144',114'/0L@62BAR,54'/100L,52'/30L,	0.00	432.25	21.148	122
								42',36'/150L				0
104	9/29/89	9/30/89	172	170 TO 154	400	152 TO 28	200	158'/50L,156'/160L,	0.00	537.49	26.297	140
106	9/29/89	10/02/89	210	208 TO 202	400	200 TO 26	200	208-202,194-190,184,180,178,168,162,156,152,	0.00	533.25	26.090	182
								146-142'/0L@60+BAR,70'/100L,				0
108	10/07/89	10/10/89	242	240 TO 172	400	170 TO 10	200	REFUSAL:236,230,224,218-208,202,198,186,164,	141.26	736.31	268.208	229
								162'/0L,20'/52L				0
110	10/05/89	10/06/89	274	272 TO 262	400	260 TO 24	200	150'/0L REFUSED @ 55BAR	0.00	918.18	44.923	246
112	10/07/89	10/11/89	276	274 TO 266	400	262 TO 10	200	REFUSAL:216,190,86,86'/0L,226-224'/30L,222'/	256.53	656.85	357.752	260
								50L,220'/30L,218'/120L,214'/60L				0
114	10/05/89	10/10/89	306	304 TO 18	200	TO		REFUSAL:290,66,38'/0L,68'/30L	360.21	629.66	419.392	286
116	10/03/89	10/05/89	316	314 TO 308	400	306 TO 42	200	REFUSALS:308,304,302,296,294,200,186,164,154.	0.00	889.93	152.864	270
								152,150,138,136'/0L				0
118				TO		TO		NO SLEEVE PIPE INSTALLED TO DATE				0
120	10/03/89	10/05/89	382	382 TO 374	400	372 TO 12	200	REFUSALS:382,140,122,104'/0L,98'/40L	0.00	1300.99	63.652	368
122	10/04/89	10/07/89	380	378 TO 370	400	368 TO 20	200	REFUSALS:180,174,164,126-120,114'/0L,	0.00	1240.96	60.715	356
								118'/100L,20'/40L				0
124	10/04/89	10/06/89	380	378 TO 372	400	370 TO 12	200	REFUSALS:196,186,172,168,138,132-122,118'/0L,	0.00	1246.61	60.992	364
								178'/20L,116'/30L				0
126	10/09/89	10/13/89	382	380 TO 352	400	350 TO 20	200	REFUSAL:330,328,218-214,218-214,198,196,180,				353
								174-144,138-132,120/0L,252'/160L,128'/160L,				0
								126'/20L,124'/60L,122'/20L				0
128	10/07/89	10/13/89	356	354 TO 202	400	200 TO 102	200	REFUSAL:320,224,216,204,200,194,170,166,164,	1280.87	211.89	237.780	250
				NO.128 THIRD REACH		100 TO 12	150	156-150,104,86,10-12'/0L,328'/20L,292'/200L,				88
								266'/50L				0
130	10/10/89	10/18/89	198	196 TO 178	400	176 TO 120	200	REFUSAL:196,194,186-184,180-168,154,142,138,	0.00	190.70	9.330	74
				NO.130 THIRD REACH		118 TO 10	150	106;18-10'/0L	0.00	259.56	290.630	108
132	10/10/89	10/18/89	180	178 TO 166	400	164 TO 120	200	REFUSAL:172,168,164,154-144,118,96;94'/0L	0.00	183.64	8.985	56
				NO.132 THIRD REACH		118 TO 10	150		0.00	275.45	290.620	108
134				TO		TO		NO SLEEVE PIPE INSTALLED TO DATE				0
								GROUT TOTALS CF:	2040.84	10617.00	-------	3990 LF OF HOLE GROUTED
								TOTAL GROUT U/S LINE:	12657.84 CF		2356.841 USG SILICATE	

TABLE IX-3

PRIMARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LINE ****SECOND PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
200	9/29/89	10/02/89	160	158 TO	152	400	150 TO	22	200	152'/14L,150'/40L,132-122'/0L@50BAR	0.00	458.39	22.427	134
202	9/29/89	10/02/89	174	172 TO	164	400	162 TO	28	200	160,144,120,66,58,56'/0L@60+BAR,60'/100L,	0.00	487.70	23.961	142
										38'/110L				0
204	9/25/89	10/02/89	212	210 TO	204	400	202 TO	82	200	174,168,162,156,146'/0L@60+BAR,82'/20L	0.00	445.67	21.805	126
206	10/04/89	10/05/89	240	238 TO	230	400	228 TO	44	200	REFUSALS:234,226,224,214,192,182-166,162-140,	0.00	503.59	24.638	192
										134,132,124'/0L,164,138'/100L,44'/60L				0
208	10/05/89	10/06/89	254	252 TO	170	400	168 TO	20	200	REFUSALS:250,220,202,166,164,124'/0L,	0.00	830.25	40.621	230
										218'/50L,212,210'/10L,208,200-196,192-184,				0
										180,156'/20L,152,/10L,150'/50L,146-142'/10L,				0
										132'/30L,20'/70L				0
210	10/05/89	10/06/89	282	280 TO	172	400	270 TO	54	200	REFUSALS:186,184,174,164,162,158,154-150,144,	0.00	739.49	323.527	224
										142'/0L,172'/60L,170'/20L,168'/30L,120'/30L				0
212	10/05/89	10/07/89	308	306 TO	298	400	296 TO	38	200	REFUSAL:170'/0L,80'/30L,52'/10L,38'/190L	0.00	968.33	47.376	256
214	10/05/89	10/11/89	330	328 TO	320	400	318 TO	34	200	REFUSAL:206,198-192,188,182-158,152,150,	490.28	402.59	19.697	292
										146-136,130'/0L				0
216	10/03/89	10/05/89	380	378 TO	360	400	358 TO	42	200	REFUSALS:196,164,162,156,154'/0L	0.00	1230.37	60.197	334
218	10/03/89	10/05/89	380	378 TO	360	400	358 TO	14	200	REFUSALS:378,360,182,164,162'/0L,204'/40L,	0.00	1292.52	63.238	362
										172'/100L,158'/20L,142'/40L				0
220	10/03/89	10/05/89	384	382 TO	374	400	372 TO	20	200	REFUSALS:379,200,196,192,190,184-180,176,174,	0.00	1161.85	56.845	360
										168,166,156-152,144,136,134,120'/0L,172'/60L,				0
										170'/40L,70'/90L				0
222	10/06/89	10/10/89	384	382 TO	372	400	366 TO	74	200	REFUSAL:278,242,206,204,146,130,126;124'/0L,	0.00	967.62	47.342	306
										390'/10L,276'/30L,274'/40L,270'/90L,268'/20L,				
										246'/20L,244'/30L,240-238'/20L,236'/120L,				
										228'/100L,176,154'/100L,174,172,160-156'/50L,				
										148'/100L,144,136'/50L,132'/120L;128'/130L				
224	10/10/89	10/14/89	374	372 TO	364	400	362 TO	122	200	REFUSAL:334,332,210,190,188,180,176,162,160,	824.60	228.84	290.620	248
					NO.224 THIRD REACH		118 TO	10	150	156,154,138,30-10'/0L,362,354,350'/50L,				108
										34'/30L				0
226	10/06/89	10/11/89	366	364 TO	358	400	356 TO	28	200	REFUSAL:196,176-170,164,158,136,50'/0L	614.48	296.64	14.514	334
											0.00	254.27	280.050	0
228	10/11/89	10/17/89	292	290 TO	192	400	190 TO	110	200	REFUSAL:282-274,264,260,246,240-236,230,228,	391.29	180.46	8.829	178
					NO.228 THIRD REACH		138 TO	12	150	220-204,186,184,172,160,152,138,136,132,128,		246.14	285.340	96
										112,108,98,24-12'/0L,264'/200L;26'/120L				0
230	10/06/89	10/10/89	194	192 TO	174	400	172 TO	24	200	REFUSAL:186-168,160-156,98,78,76'/0L,24'/150L	0.00	506.77	355.110	166
232	10/10/89	10/12/89	172	170 TO	162	400	160 TO	12	200	REFUSAL:156,154'/0L,170-168'/20L,166'/30L,	0.00	518.42	416.718	156
										164'/10L,162'/20L,12'/160L				0
234				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	2310.65	11719.91	-------	4254 LF OF HOLE GROUTED
										TOTAL GROUT D/S LINE:	14030.56 CF		2402.755 USG SILICATE	

TABLE IX-4

FRESSURE GROUTING SUMMARY : CONTRACT NC. DACW67-88-C-0047 ******DATA DATE: 12/06/89 *********

PRIMARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****THIRD PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT / REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
100	10/12/89	10/13/89	150	120 TO	29	200	TO			REFUSAL:120-118,106'/0L,104'/40L,20'/160L	0.00	332.66	364.600	100
102	10/13/89	10/13/89	162	120 TO	16	150	TO			REFUSAL:94,58,50'/0L,72'/20L,52'/10L	0.00	255.33	290.620	104
104	10/13/89	10/13/89	172	120 TO	28	150	TO			NONE, 10-20 BAR ATTAINED	0.00	248.97	290.620	92
106	10/13/89	10/14/89	210	120 TO	12	200	TO			24'/120L @ 25 BAR	0.00	385.64	356.670	108
108	10/12/89	10/12/89	242	240 TO	210	400	TO			REFUSAL:236,232-226,222-210'/0L @ 70 BAR	56.50	0.00	0.000	30
110	10/11/89	10/13/89	274	272 TO	240	400	262 TO	152	200	REFUSAL:272,268,22-12'/0L,252'/30L,172'/150L,	198.62	174.81	8.553	80
				NO.110 THIRD REACH			120 TO	12	150			259.56	295.904	108
112	11/22/89	11/23/89	276	150 TO	12	200	TO			REFUSAL:148-144,140-124,120,116,114,106,98-92, 78,74,60-52,32,30,20-12'/0L,22'/140L	0.00	252.15	12.337	138
114	10/12/89	10/13/89	306	200 TO	10	100	TO			REFUSAL:192-186,180-172,168-158,130,108,78,	0.00	254.27	12.440	190
										72,70,60,56,54,38'/0L				0
116	10/13/89	10/13/89	316	130 TO	22	200	TO			REFUSAL:128-112,102,100,86,82,80,72,56,48,	256.38	0.00	0.000	108
										46'/0L,22'/60L				0
118				TO			TO			NOT INSTALLED TO DATE				0
120	10/13/89	10/16/89	382	120 TO	10	150	TO			NONE	0.00	296.64	295.904	110
122	10/18/89	10/20/89	380	150 TO	12	150	TO			REFUSAL:140,144-136,130,128,116,94;92'/0L	0.00	312.54	369.880	138
124	10/17/89	10/18/89	380	120 TO	12	200	TO			REFUSAL:120'/0L	0.00	381.40	285.340	108
126	10/10/89	10/13/89	392	380 TO	352	400	350 TO	20	200	REFUSAL:330,328,218-214,198,196,180,174-144,	0.00	317.83	15.550	358
										138-132;120'/0L,252'/100L,128'/160L,126'/20L,				0
										124'/60L,122'/20L				0
128			358	TO			TO							0
130	10/19/89	10/19/89	198	92 TO	70	200	TO			NONE	0.00	84.76	4.147	22
132	10/19/89	10/20/89	180	92 TO	20	200	TO			REFUSAL:84;76'/0L	0.00	247.20	12.095	72
134				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	511.70	3803.76	-------	1866 LF OF HOLE GROUTED
										TOTAL GROUT U/S LINE:	4315.46 CF		2614.660 USG SILICATE	

TABLE IX-5

PRIMARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LINE ****THIRD PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
200	10/12/89	10/12/89	160	120 TO	90	200	TO			REFUSAL:108'/0L	0.00	105.94	118.900	30
202			174	TO			TO							0
204	10/12/89	10/13/89	212	120 TO	28	200	TO			REFUSAL:110,58,42'/0L	0.00	310.77	326.300	92
206	10/13/89	10/16/89	210	100 TO	12	200	TO			REFUSAL:64,46,42,;26-12'/0L,28'/180L	0.00	239.43	253.632	88
208	10/17/89	10/18/89	254	130 TO	12	200	TO			REFUSAL:110,102,90,64,58,48;40-12'/0L,	0.00	257.80	298.550	118
										126'/160L,68;60'/50L,42'/40L				
210	10/12/89	10/12/89	282	[illegible]	252	400	250 TO	230	200	REFUSAL:272,270,266,234'/0L	0.00	240.14	11.749	48
212	10/12/89	10/13/89	300	[illegible]	218	200	130 TO	22	200	REFUSAL:294,292,286,284,258,224-218,126'/0L,	233.00	364.45	420.080	190
										108'/30L,50'/20,28'/160L,22'/90L				0
214	10/18/89	10/18/89	330	118 TO	12	150	TO			REFUSAL:26-12'/0L,42'/50L	0.00	236.61	285.340	106
216	10/16/89	10/20/89	380	378 TO	360	1000	358 TO	330	405	REFUSAL:368,362-358,348,342-338,332,308,300,	0.00	1097.23	53.683	46
				NO. 216 THIRD REACH			328 TO	44	200	288,276,274,266,250,216,212,208,189,178,168,				
										164-160,154-126,118,114,96;46'/0L,376'/630L,				
										298,196;182'/100L,198'/20L,192'/80L;44'/40L				
218	10/17/89	10/19/89	380	378 TO	370	400	120 TO	12	200	REFUSAL:376-370,26-12'/0L,66'/100L;36'/170L	0.00	348.56	290.620	116
220			384	TO			TO							0
222	10/11/89	10/11/89	384	100 TO	12	200	TO			REFUSAL:44,40'/0L	0.06	303.71	361.960	88
224			366	TO			TO							0
226	10/17/89	10/18/89	366	110 TO	24	200	TO			REFUSAL:110'/0L	0.00	300.18	14.686	36
228	10/23/89	10/23/89	294	120 TO	54	200	TO			NONE	0.00	240.14	293.263	66
230	11/10/89	11/11/89	194	192 TO	176	200	150 TO	16	200	REFUSAL:188-176,148-126,104,68,66'/0L,16'/140L	386.34	0.00	0.000	150
232			172	TO			TO							0
234				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	619.42	4044.96	-------	1224 LF OF HOLE GROUTED
										TOTAL GROUT D/S LINE:	4654.38 CF		2728.763 USG SILICATE	

TABLE IX-6

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-88-C-0047 ******DATA DATE: 12/06/89 *********
PRIMARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****FOURTH PASS**** ****FIFTH PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: / REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
100	10/20/89	10/23/89	150	100 TO	16	200	TO			REFUSAL:96;94'/0L	0.00	289.58	14.168	84
	11/21/89	11/22/89		150 TO	12	200 **FIFTH PASS**				REFUSAL:150-138,130,128,124-116,110-106,100, 96-86,74,68,16-12'/0L,80'/60L,32'/100L,30'/ 20L,28'/10L ***FIFTH PASS***	0	268.04	13.114	138
102			162	TO			TO							0
104			172	TO			TO							0
106	11/22/89	11/22/89	210	100 TO	12	200	TO			REFUSAL:96,94,90,80,66,64,60,56,64,32;28'/0L, 78'/20L,68'/30L,16'/50L,14'/20L,12'/20L	0.00	206.24	10.090	88
108			242	TO			TO							0
110			274	TO			TO							0
112			276	TO			TO							0
114			306	TO			TO							0
116			316	TO			TO							0
118				TO			TO							0
120	10/23/89	10/24/89	382	120 TO	22	260	TO			REFUSAL:106,74,68-64;50'/0L,120;112'/100L	0.00	303.71	14.859	98
122	11/22/89	11/27/89	380	150 TO	12	200	TO			REFUSAL:114-106,102-92,86,74,70,64,60-54,48, 42-38,18-12'/0L,104'/40L	0.00	298.06	14.583	138
124	10/19/89	10/20/89	380	48 TO	14	200	TO			NONE	0.00	127.13	6.220	34
126			382	TO			TO							0
128			358	TO			TO							0
130	11/22/89	11/27/89	198	150 TO	16	200	TO			REFUSAL:146,142-116,106,102-86,74,72,58,54, 48;18'/0L,108'/50L,16'/30L	0.00	250.03	12.233	134
132	11/29/89	11/29/89	180	150 TO	24	200	TO			REFUSAL:150-146,142,140,136,134,130-106,92,90, 76-72,68,66,62,60'/0L,94'/20L,24'/40L	0.00	235.20	11.507	126
134				TO			TO			NO SLEEVE PIPE INSTALLED TO DATE				0
										GROUT TOTALS CF:	0.00	1977.99	-------	840 LF OF HOLE GROUTED
										TOTAL GROUT U/S LINE:	1977.99 CF		96.774 USG SILICATE	

TABLE IX-7

PRIMARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LINE ****FOURTH PASS**** ****FIFTH PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE /BOTTOM/	REACH ONE /TOP/	/LITRES PER STG/	REACH TWO /BOTTOM/	REACH TWO /TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
200	11/22/89	11/22/89	160	150 TO	12	200	TO			REFUSAL:88,80,72,68'/0L,70'/40L,34'/130L	0.00	458.03	22.410	138
202			174	TO			TO							0
204			212	TO			TO							0
206	10/20/89	10/20/89	240	46 TO	38	200	TO			REFUSAL:42;38'/0L	0.00	49.44	2.419	16
	11/22/89	11/22/89		100 TO	14	200	**FIFTH PASS**			REFUSAL:98-94,90,82-78,70-62,56,50-42,38,34, 28-12'/0L,88;86'/20L,52'/140L,58'/60L ***FIFTH PASS***	0	86.17	4.216	86
208	10/23/89	10/23/89	254	92 TO	88	200	TO			REFUSAL:90'/0;88'/100L	0.00	10.59	0.518	4
210			282	TO			TO							0
212	11/22/89	11/22/89	308	120 TO	48	200	TO			REFUSAL:120,112,86'/0L,92'/50L,70'/150L,56'/ 130L,48'/160L	0.00	229.19	11.213	72
214			330	TO			TO							0
216			380	TO			TO							0
218			388	TO			TO							0
220			384	TO			TO							0
222	11/22/89	11/27/89	384	150 TO	22	200	TO			REFUSAL:150'/0L,138'/45L,22'/80L	0.00	442.14	21.632	128
224			366	TO			TO							0
226	10/23/89	10/24/89	366	110 TO	66	200	40 TO	24	200	REFUSAL:88;40'/0L	0.00	211.89	245.707	60
228	11/10/89	11/10/89	294	292 TO	50	200	TO			REFUSAL:290-272,266-254,248,242-198,192-168, 162,166,156-78,74-66'/0L,196'/30L	135.26	0.00	0.000	242
230			194	TO			TO							0
232			172	TO			TO							0
234				TO			TO							0
										GROUT TOTALS CF:	135.26	1487.45	-------	746 LF OF HOLE GROUTED
										TOTAL GROUT D/S LINE:	1622.71 CF		308.115 USG SILICATE	

TABLE IX-8

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-85-C-0047 ******DATA DATE: 12/14/89 *********

SECONDARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****FIRST PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

				PRE-PROGRAMMED AMOUNTS OF GROUT										
/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	/ REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
101	11/02/89	11/07/89	158	156 TO	148	400	146 TO	12	200	REACH1 20-27 BARS,REACH2 3-24 BARS	169.51	381.40	692.210	142
103	11/02/89	11/08/89	170	168 TO	160	400	158 TO	29	200	REFUSAL:150'/100L,114'/10L;108'/0L.	208.36	311.12	416.110	138
105	11/02/89	11/09/89	176	174 TO	166	400	164 TO	34	200	REFUSAL:76,60;58'/0L.	233.08	275.45	234.610	138
107	11/02/89	11/13/89	242	240 TO	232	400	230 TO	12	200	REFUSAL:122,56,46,18;12'/0L,32'/140L	353.15	456.97	335.790	226
109	11/03/89	11/10/89	258	256 TO	248	400	246 TO	26	200	REFUSAL:112,110,92,80,54-60'/0L,196'/50L	404.35	395.52	190.093	228
111	11/03/89	11/14/89	268	266 TO	258	400	256 TO	16	200	REFUSAL:156,152,150,102,88,78,76'/0L,50'/20L	430.84	438.61	208.180	248
113	11/06/89	11/10/89	298	296 TO	288	400	286 TO	18	200	REFUSAL:148,126,124,32;30'/0L,150'/30L, 142'/130L	550.91	429.43	236.328	276
115	11/06/89	11/14/89	314	312 TO	304	400	302 TO	18	200	REFUSAL:234'/100L,182'/30L;122,105-102,54; 34'/0L,18'/30L	483.81 114.07	424.84	425.360	292
117	11/06/89	11/15/89	354	352 TO	344	400	342 TO	26	200	REFUSAL:122,102,62,60;28'/0L,26'/10L	776.93	374.69	235.130	324
118	11/07/89	11/16/89	384	382 TO	348	400	338 TO	20	200	REFUSAL:120,114,92,90,86;44'/0L. 118 DRILLED LATE. GROUTED WITH SECONDARIES.	1080.63	317.83	310.120	360
119	11/06/89	11/13/89	384	382 TO	360	400	358 TO	12	200	REFUSAL:186;138'/0L,16'/100L	896.99	494.16	23.688	368
121	11/08/89	11/15/89	384	382 TO	374	400	372 TO	16	200	REFUSAL:116,68,46-40;20'/0L,80'/170L;22'/50L	960.56	318.54	118.600	364
123	11/08/89	11/14/89	382	380 TO	372	400	376 TO	12	200	NONE	847.55	494.76	24.207	366
125	11/09/89	11/15/89	382	360 TO	372	400	370 TO	14	200	REFUSAL:98,76;22'/0L,58'/100L,56'/50L	847.55	457.33	509.380	364
127	11/09/89	11/13/89	372	370 TO	356	400	354 TO	10	200	REFUSAL:52'/0L,32'/30L	833.43	488.40	23.896	358
129	11/10/89	11/16/89	252	246 TO	196	400	194 TO	12	200	REFUSAL:140,124,122,104,100,64,58;20'/0L, 78'/20L,76'/30L;22'/10L	480.28	418.83	236.460	226
131	11/10/89	11/14/89	188	186 TO	178	400	176 TO	10	200	REFUSAL:22 /160L,20-14'/10L,12'/100L,10'/150L	162.45	467.92	22.893	174
											-------	-------		-------
										GROUT TOTALS CF:	9834.45	6935.80	-------	4592.00 LF OF HOLE GROUTED
										TOTAL GROUT U/S LINE:	16770.25 CF		3550.85 USG SILICATE	

TABLE IX-9

SECONDARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LIN****FIRST PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	REACH ONE /BOTTOM/	REACH ONE /TOP/	/LITRES PER STG/	REACH TWO /BOTTOM/	REACH TWO /TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
201	11/02/89	11/08/89	170	168 TO	160	400	158 TO	120	200	NONE		346.00	359.580	46
				NO.201 THIRD REACH:			120 TO	22	200	REFUSAL:86'/0L	211.89			98
203	11/02/89	11/09/89	180	178 TO	170	400	168 TO	24	200	REFUSAL:112,52,50'/0L	247.20	317.83	294.850	152
205	11/02/89	11/14/89	238	236 TO	228	400	226 TO	12	200	REFUSAL:134'/0L,34,24,18;12'/100L,32;26'/50L, 30,28,20;16'/10L,22'/180L,14'/150L	339.02	433.31	21.200	222
207	11/02/89	11/15/89	254	252 TO	244	400	242 TO	12	200	REFUSAL:136,134,126,124,64,62,34;32'/0L,138'/ 100L,100'/130L,68'/80L,52,42,14'/150L,30'/10L 28,12'/50L,26'/40L,24,20'/20L,18'/170L	395.52	385.64	18.868	238
209	11/03/89	11/15/89	286	284 TO	276	400	274 TO	12	200	REFUSAL:170,138;134'/0L,148;26'/50L,146'/80L, 144'/170L,140'/100L,132'/160L,30,28;16'/10L, 18'/100L,14'/40L,12'/99L	501.47	426.25	20.855	270
211	11/03/89	11/15/89	302	300 TO	292	400	290 TO	8	200	REFUSAL:38;32'/100L,30;22'/50L,24'/90L,20;14'/ 50L,16'/70L,12;10'/110L,8'/20L	561.50	458.03	22.410	290
213	11/06/89	11/15/89	338	336 TO	328	400	326 TO	48	200	208'/150L,194'/120L,152'/150L,52'/50L,48'/100L	678.75	416.36	20.544	286
				NO.213 THIRD REACH:			30 TO	12	200	16'/10L,12'/30L REFUSED FURTHER TAKE				
215	11/06/89	11/15/89	348	346 TO	338	400	336 TO	12	200	NONE	734.55	494.41	24.189	332
217	11/06/89	11/15/89	382	380 TO	364	400	362 TO	12	200	REFUSAL:166,146,144;112'/0L,114'/180L,30'/10L 28,26'/30L,24'/120L	868.75	443.91	21.719	366
219	11/07/89	11/15/89	392	390 TO	374	400	372 TO	12	200	30'/20L,28;26'/40L,24;22'/30L,20;14'/70L, 18'/60L REFUSED FURTHER TAKE	840.49	443.55	21.701	366
221	11/07/89	11/15/89	380	378 TO	372	400	370 TO	12	200	REFUSAL:38-32'/0L,40'/90L,22'/140L,18,16'/40L 14'/60L 38-32' MAY HAVE BEEN SKIPPED	833.43	443.55	21.701	364
223	11/08/89	11/15/89	380	378 TO	372	400	370 TO	12	200	REFUSAL:150;76'/0L,154'/130L,148'/150L,118'/ 110L,28'/40L,26'/30L,24'/60L,22'/130L,18'/140L 14'/60L	829.90	449.20	21.978	364
225	11/09/89	11/15/89	384	382 TO	376	400	374 TO	12	200	REFUSAL:144-136'/0L,282'/110L,148'/100L,146'/ 60L,136'/130L,26,20;18'/50L,16'/80L,14'/20L	844.38	428.72	20.976	368
227	11/09/89	11/15/89	336	334 TO	302	400	300 TO	12	200	REFUSAL:134'/0L,302'/20L,24'/50L,22-20'/150L, 18'/40L,16-14'/90L,12'/120L	763.50	462.27	22.617	320
229	11/09/89	11/17/89	202	200 TO	194	400	192 TO	12	200	REFUSAL:34;32'/0L,124'/30L,120'/140L,106'/60L 66;36'/150L,14'/10L	204.83	456.97	22.358	186
231	11/10/89	11/15/89	182	180 TO	12	200	TO			REFUSAL:144,136,124,80;78'/0L,66'/100L	105.94	455.56	22.289	168
										GROUT TOTALS CF:	8961.12	6861.64	-------	4436.00 LF OF HOLE GROUTED
										TOTAL GROUT D/S LINE:	15822.76 CF		598.26 USG SILICATE	

TABLE IX-10

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-89-C-0047 ******DATA DATE: 12/14/89 *********

SECONDARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****SECOND PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE /BOTTOM/	REACH ONE /TOP/	/LITRES PER STG/	REACH TWO /BOTTOM/	REACH TWO /TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
101	11/15/89	11/17/89	158	150 TO	12	200	TO			REFUSAL:150-142,130-102,82,70-66,58,48,38,26-22,16;14'/0L,50'/100L,12'/80L 12-30' DONE 1ST	0.00	260.62	12.751	138
103	11/18/89	11/20/89	170	150 TO	12 12-30' DONE FIRST	200	TO			REFUSAL:150,148,136,122-116,108,84,82,60,58,46,16'/0L,78'/50L,44'/10L,30'/120L	0.00	338.32	16.552	138
105	11/17/89	11/18/89	176	150 TO	12	200	TO			REFUSAL:140,136,130,128,120,116-110,100,66,42;40'/0L	0.00	402.59	19.697	138
107	11/15/89	11/20/89	240	150 TO	12	200	TO			REFUSAL:136,120,20;16'/0L,32-28'/20L,26'/50L,24'/60L,18'/10L	0.00	430.13	21.045	138
109	11/15/89	11/16/89	258	150 TO	12 12-30' DONE FIRST	200	TO			REFUSAL:150-144,138,112,110,92,80,72,64,56,38,26;12'/0L,136'/20L,46;22'/150L,40'/50L	0.00	373.28	18.263	138
111	11/15/89	11/18/89	268	150 TO	12	200	TO			REFUSAL:148-144,132,126,122,120,22'/0L,150'/30L,114'/120L,117'/30L,110'/40L,106'/20L,104'/60L,32'/20L,26'/110L,24'/160L,20'/160L,16'/120L,14'/60L,12'/30L	0.00	379.99	18.591	138
113	11/15/89	11/16/89	298	150 TO	12 12-30' DONE FIRST	200	TO			REFUSAL:140,130,108,106,96,88,78,68,54,46,38'/0L,64'/100L,16'/20L,14'/10L,12'/70L	0.00	395.52	19.351	138
115	11/16/89	11/17/89	314	150 TO	12	200	TO			REFUSAL:148,146,136-116,96,80,76;74'/0L	0.00	367.98	18.034	138
117	11/10/89	11/20/89	354	352 TO	310	500	150 TO	12	200	REFUSAL:348-340,144,138-134,112-104;74'/0L,96,18;16'/50L 12-30' DONE BEFORE 150-30'	282.52	407.89	19.956	180
119	11/21/89	11/22/89	384	150 TO	12	200	TO			REFUSAL:148,144,142,136,126,80,72,58,56,52,40'/0L,124'/20L,90'/60L,54'/60L,12'/150L	0.00	393.70	19.507	138
119	11/10/89	11/18/89	384	362 TO	352	800	350 TO	330	500	REFUSAL:370'/0L,372'/200L	568.57	0.00	0.000	50
				THIRD REACH NO. 119			150 TO	30	200	REFUSAL:144,138-124,120,109,98,94,80,60,58,54-50,44'/0L,142'/100L,82'/40L,36'/30L	0	268.04	13.114	120
121	11/20/89	11/21/89	334	150 TO	12	200	TO			REFUSAL:149,126,124,120-116,112-104,100-96,80,78,70-66,58,52,50'/0L,146;102'/30L,54'/100L	0.00	323.49	15.827	138
123	11/17/89	11/17/89	382	150 TO	12	200	TO			REFUSAL:152,148,144,132,130,124,120-106,100,94,86,84,80,78,70-66,54'/0L,96'/150L,88;48'/100L	0.00	316.07	15.464	138
125	11/16/89	11/17/89	362	150 TO	12	200	TO			REFUSAL:148-132,122-116,88,84,82,74-68,54,52,34,28,16'/0L,150'/50L,136'/20L,128'/180L,90'/60L,86'/20L	0.00	308.30	15.084	138
127	11/21/89	11/22/89	372	150 TO	12	200	TO			REFUSAL:146,138-134,120,116-110,94-86,64,62,44;36'/0L,118'/20L	0.00	360.92	17.658	138
129	11/09/89	11/16/89	242	240 TO	196	400	194 TO	12	200	REFUSAL:140,124,122,104,100,64,58;20'/0L,78'/20L,76'/30L,22'/10L	0.00	399.06	19.524	226
131	11/16/89	11/20/89	186	150 TO	12	200	TO			REFUSAL:140,146,136,130,128,110,106,100,72-64,46,22'/0L,150;138'/30L,118'/50L,108'/100L,30'/70L,12'/30L	0.00	357.83	17.468	138
										GROUT TOTALS CF:	851.09	6087.92	-------	2508.00 LF
										TOTAL GROUT U/S LINE:	6939.01 CF		285.11 USG SILICATE	

TABLE IX-11

SECONDARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LIN****SECOND PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: / REACH ONE / /BOTTOM/	/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
201	11/15/89	11/16/89	170	150 TO	12	200	TO			REFUSAL:150-146,140-132,128-116,66,38;32'/0L 130'/50L,34'/100L,30'/160L,28'/110L,26'/60L, 24'/70L,22'/10L,16'/140L;12'/100L 12-30' 1ST	0.00	331.96	16.241	138
203	11/15/89	11/20/89	178	150 TO	12	200	TO			REFUSAL:150,146'/0L,138'/60L,18'/100L,16'/190L, 14'/50L,12'/180L	0.00	459.09	22.461	138
205	11/17/89	11/21/89	238	150 TO	12	200	TO			REFUSAL:150-140;70'/0L,72'/150L,16'/50L	0.00	437.90	21.425	138
207	11/20/89	11/22/89	254	150 TO	12	200	TO			REFUSAL:136-132,124,120'/0L,138;106'/50L	0.00	448.5	21.943	138
209	11/21/89	11/22/89	286	150 TO	24	200	TO			REFUSAL:150,146-140,132,126-86,62,60,48'/0L, 148;128'/50L,24'/20L	0.00	216.13	10.574	126
211	11/17/89	11/20/89	302	150 TO	18	200	TO			REFUSAL:106'/140L;32'/100L @ 10-12 BAR	0.00	467.57	22.876	132
213	11/20/89	11/21/89	338	150 TO	36	200	TO			REFUSAL:144-140'/0L,150'/30L,68'/150L	0.00	380.69	18.626	114
215	11/22/89	11/22/89	348	150 TO	12	200	TO			REFUSAL:150,136,126,106,98,96,52-46,32'/0L, 94'/30L,78'/11L	0.00	404.04	19.768	138
217	11/20/89	11/22/89	382	150 TO	16	200	TO			REFUSAL:150,148,114'/0L,144'/20L,120'/100L, 16'/100L	0.00	445.67	21.805	134
219	11/22/89	11/22/89	382	150 TO	34	200	TO			REFUSAL:148,146,134,60'/0L,62'/40L	0.00	392.81	18.729	116
221	11/22/89	11/22/89	380	150 TO	24	200	TO			REFUSAL:136,134,128-122,54'/0L,56;24'/100L	0.00	395.52	19.351	126
223	11/21/89	11/22/89	380	150 TO	12	200	TO			REFUSAL:150,146,142,138,136,132,130,126-114, 100,94,90-86,76,68-62'/0L,140'/90L,134'/50L	0.00	315.71	15.447	138
225	11/20/89	11/21/89	394	150 TO	16	200	TO			REFUSAL:150-136,98,68'/0L	0.00	409.65	20.043	134
227	11/22/89	11/23/89	336	150 TO	20	200	TO			REFUSAL:136,60'/0L	0.00	452.03	22.116	130
229	11/21/89	11/21/89	202	150 TO	12	200	TO			REFUSAL:150,148,138,134,130,116-108,102-98; 94'/0L,128'/50L	0.00	383.16	18.747	138
231	11/21/89	11/22/89	182	150 TO	20	200	TO			REFUSAL:150,146,144,134,120'/0L,140'/90L,138'/ 20L,44'/50L,40'/100L,38'/50L,20'/110L	0.00	403.29	19.732	130
											-------	-------		-------
										GROUT TOTALS CF:	0.00	6333.72	-------	2108.00 LF
										TOTAL GROUT D/S LINE:	6333.72 CF		293.64 USG SILICATE	

TABLE IX-12

PRESSURE GROUTING SUMMARY : CONTRACT NO. DACW67-88-C-2947 ******DATA DATE: 12/14/89 *********

SECONDARY GROUT HOLES @ 12 FT SPACING: UPSTREAM LINE ****THIRD/FOURTH PASS****

EACH REACH IS GROUTED USING 2 FT STAGES

				PRE-PROGRAMMED AMOUNTS OF GROUT													
/HOLE NO./	/START DATE/	/FINISH DATE/	/PIPE DEPTH/	/ REACH ONE / /BOTTOM/		/TOP/	/LITRES / PER STG/	REACH TWO / /BOTTOM/		/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
101	11/27/89	11/27/89	158	100	TO	20	200		TO			REFUSAL:96,88-84,76,74,66,60,48-40,36,34'/0L	0.00	183.64	8.985	80
	11/30/89	11/30/89		30	TO	18	200					REFUSAL:28-22'/0L,18'/130L FOURTH PASS	0.00	12.36	0.605	12
103			170		TO				TO							0
105	11/27/89	11/27/89	176	100	TO	18	200		TO			REFUSAL:54,34-30;18'/0L	0.00	261.33	12.786	82
	11/30/89	11/30/89	176	110	TO	12	200	20	TO	12	200	REFUSAL:50-42'/0L FOURTH PASS/PARTLY A FIFTH	0.00	353.14	17.278	106
107			240		TO				TO							0
109	11/28/89	11/28/89	258	100	TO	22	200		TO			REFUSAL:94-90,80'/0L	0.00	254.27	12.440	78
	12/01/89	12/01/89		30	TO	24	200	26	TO	20	200	REFUSAL:24'/50L,20'/170L FOURTH PASS	0.00	28.96	1.417	6
111			268		TO				TO							0
113	12/01/89	12/01/89	298	20	TO	14	200		TO			NONE	0.00	28.25	1.382	6
115			314		TO				TO							0
117	11/21/89	11/21/89	354	334	TO	302	400		TO			REFUSAL:334-322,314-306;302'/0L UP TO 60 BAR	0.00	56.50	2.764	32
	12/01/89	12/01/89	354	30	TO	26	200	18	TO	12	200	REFUSAL:26'/60L FOURTH PASS	0.00	44.50	2.177	10
118			384		TO				TO							0
119	11/15/89	11/16/89	384	360	TO	302	800		TO			REFUSAL:338,336'/0L,360'/20L	0.00	763.50	37.355	58
	11/28/89	11/28/89		100	TO	18	200		TO			REFUSAL:NONE FOURTH PASS	0.00	296.64	14.514	82
121			384		TO				TO							0
123	11/28/89	11/28/89	362	100	TO	24	200		TO			REFUSAL:96,82,78,70,65-60,48,46,40-34,28'/0L 80'/20L	0.00	163.15	7.982	76
125			382		TO				TO							0
127	11/30/89	11/30/89	372	110	TO	18	200		TO			REFUSAL:68'/0L	0.00	324.90	15.896	92
	11/31/89	11/31/89		30	TO	28	200	26	TO	18	200	REFUSAL:30'/0L,18'/10L	0.00	14.48	0.708	4
129			242		TO				TO							0
131			186		TO				TO							0
													-------	-------		-------
												GROUT TOTALS CF:	0.00	2785.62	-------	724.00 LF
												TOTAL GROUT U/S LINE:	2785.62 CF		126.70 USG SILICATE	

TABLE IX-13

SECONDARY GROUT HOLES @ 12 FT SPACING: DOWNSTREAM LIN****THIRD/FOURTH PASS****
EACH REACH IS GROUTED USING 2 FT STAGES

/HOLE NO./	/START DATE/	/FINISH DATE/	/BOTTOM DEPTH/	PRE-PROGRAMMED AMOUNTS OF GROUT: REACH ONE /BOTTOM/	/TOP/	/LITRES PER STG/	REACH TWO /BOTTOM/	/TOP/	/LITRES PER STG/	/DEVIATIONS FROM PRE-PROGRAMMED AMOUNTS REMARKS/	/ GROUT TOTALS / (W/O SIL)	(W/ SIL)	/USG SIL/	/GROUTED LENGTH/
201	11/27/89	11/28/89	170	114 TO	36	200	TO			REFUSAL:100,104,94,84,58'/0L,36'/40L	0.00	241.55	11.818	78
	11/30/89	11/30/89		30 TO	26	200	20 TO	20	200	REFUSAL:26'/70L FOURTH PASS	0.00	23.66	1.158	4
203	11/27/89	11/27/89	178	138 TO	126	200	116 TO	18	200	REFUSAL:130,134-126,116-106,80,74-70,64,54,52,	0.00	244.73	11.974	110
										24;22'/0L,32'/110L,66'/20L				0
205	11/28/89	11/28/89	238	112 TO	30	200	TO			STOPPED 30'/150L:GROUT SURFACING	0.00	294.88	14.427	82
	11/30/89	11/30/89		30 TO	30	200	20 TO	20	200	REFUSAL:30'/110L,20'/70L FOURTH PASS	0.00	6.36	0.311	0
207			254	TO			TO							0
209	11/27/89	11/27/89	262	280 TO	252	200	84 TO	70	200	REFUSAL:270,250;250'/0L,82'/0L,30'/100L	0.00	173.04	8.466	42
	THIRD REACH NO.209						40 TO	30	200					10
	11/30/89	11/30/89		30 TO	24	200	20 TO	18	200	REFUSAL:30,28,20'/0L,18'/20L FOURTH PASS	0.00	14.83	0.726	8
211	11/28/89	11/28/89	302	150 TO	58	200	TO			REFUSAL:146,142-138,106,104,100,98,76,70,68'/0L,	0.00	247.20	12.095	92
										144'/50L,58'/150L				0
213	11/27/89	11/28/89	340	336 TO	322	200	138 TO	36	200	REFUSAL:128-122,114,110,100,92'/0L	0.00	367.27	17.969	116
	11/30/89	11/30/89		30 TO	24	200	20 TO	20	200	REFUSAL:30-26'/0L,24'/40L,20'/50L FOURTH PASS	0.00	3.19	0.156	6
215	11/28/89	11/29/89	348	136 TO	28	200	TO			REFUSAL:136,132'/0L,28'/150L	0.00	372.57	18.228	108
217	11/28/89	11/29/89	362	140 TO	36	200	TO			REFUSAL:130,128,124'/0L,58'/130L,40;36'/150L	0.00	347.14	16.984	104
	11/30/89	11/30/89		30 TO	30	200	20 TO	20	200	REFUSAL:20'/30L FOURTH PASS	0.00	8.12	0.387	0
219	11/28/89	11/29/89	380	378 TO	370	400	140 TO	16	200	REFUSAL:122,118'/0L	0.00	501.47	24.535	132
221	11/28/89	11/29/89	380	120 TO	28	200	TO			REFUSAL:110,104'/0L,42'/120L	0.00	315.01	15.412	92
	11/30/89	11/30/89		30 TO	24	200	20 TO	18	200	REFUSAL:26,24'/0L FOURTH PASS	0.00	28.25	1.382	8
223	11/29/89	11/30/89	380	110 TO	94	200	82 TO	12	200	NONE	0.00	317.83	15.550	86
225	12/01/89	12/01/89	384	100 TO	70	200	62 TO	26	200	REFUSAL:90,26'/150L	0.00	243.67	11.922	66
227	11/29/89	11/30/89	336	150 TO	144	200	126 TO	116	200	REFUSAL:150,146'/0L	0.00	331.96	16.241	16
							100 TO	24	200					76
229	11/29/89	12/01/89	202	200 TO	186	200	84 TO	28	200	REFUSAL:198,196,188'/0L,200'/100L,28'/160L	0.00	235.20	11.507	70
231	11/29/89	12/01/89	182	180 TO	170	200	138 TO	24	200	REFUSAL:180-174,138-126,120,116,92,90,86,84,	0.00	313.95	15.360	124
										72'/0L,118'/10L,44'/160L,30'/180L				
														0
										GROUT TOTALS CF:	0.00	4631.87	-------	1430.00 LF
										TOTAL GROUT D/S LINE:	4631.87 CF		213.63 USG SILICATE	

TABLE IX-14

Summary of Grout Quantities Placed: DACW67-88-C-0047 Summary Date: 24 Jan 90
Case 17 Gravity Grouting
Case 19 Recompression Grouting

		CF W/O Sil	CF W/ Sil	Gals Sil	CY Bentonite (083.33#/CY)
Case 17, Gravity Grout	C/L	532.880	0.000	808.000	N/A
Case 19, Primary Install	U/S	3,626.100	0.000	30.000	148.780
	D/S	6,073.650	0.000	321.500	181.760
Case 19, Secondary Install	U/S	1,493.100	0.000	0.000	25.200
	D/S	1,922.400	0.000	0.000	35.400
First Primary Pass	U/S	479.220	14,363.220	867.371	N/A
	D/S	586.220	15,609.100	989.560	N/A
Second Primary Pass	U/S	2,040.840	10,617.000	2,356.841	N/A
	D/S	2,310.650	11,719.910	2,402.755	N/A
Third Primary Pass	U/S	511.700	3,803.760	2,614.660	N/A
	D/S	619.420	4,044.960	2,728.763	N/A
Fourth/Fifth Primary Pass	U/S	0.000	1,977.990	96.774	N/A
	D/S	135.260	1,487.450	308.115	N/A
First Secondary Pass	U/S	9,834.450	6,935.800	3,550.850	N/A
	D/S	8,961.120	6,861.640	598.260	N/A
Second Secondary Pass	U/S	851.090	6,087.920	285.110	N/A
	D/S	0.000	6,333.720	293.640	N/A
Third/Fourth Secondary Pass	U/S	0.000	2,785.620	126.700	N/A
	D/S	0.000	4,631.870	213.630	N/A
		39,978.100 CF W/O Sil	97,259.960 CF W/ Sil	18,592.529 Gals Sil	391.140 CY Bentonite OR
		1480.67 CY W/O Sil	3602.22 CY W/Sil		32,593.696 LBS Bentonite

Case 17 and Case 19 TOTAL CY GROUT: 5082.89

Case 12 Investigative Drilling and Grouting
TOTAL CY GROUT: 291.60

TOTAL ALL CASES 5374.49 CY

TABLE IX-15

LATERAL MOVEMENT OF
OUTSIDE SHOULDERS ON TOP OF DAM
(WORK PLATFORM ELEVATION 1240)

DATA DATE: 15 DEC 89

UPSTREAM MONUMENT	STATION	OUTWARD MOVEMENT AWAY FROM CENTERLINE OF DAM (+)	DOWNSTREAM MONUMENT	STATION	OUTWARD MOVEMENT AWAY FROM CENTERLINE OF DAM (+)
US-10	11+36	-0.2 "			
			DS-10	11+50	NR
US-09	12+00	0.0 "	DS-09	12+00	NR
US-08	12+50	0.2 "	DS-08	12+50	-0.5 "
US-07	13+00	1.4 "	DS-07	13+00	0.0 "
US-06	13+50	4.2 "			
			DS-06	13+65	4.1 "
US-05	14+00	10.1 "			
			DS-05	14+20	NR
US-04	14+50	12.6 "			
			DS-04	14+66	9.7 "
US-03	14+85	13.0 "			
			DS-03	15+10	10.1 "
US-02	15+50	10.2 "			
			DS-02	15+70	7.1 "
US-01	16+07	3.6 "			
			DS-01	16+20	0.4 "
US-01A	16+57	-0.2 "			
			DS-01A	16+72	-0.5 "
US-01B	17+05	0.0 "			
			DS-01B	17+27	1.1 "

NR = NO RECORD

TABLE IX-16

SECTION X

DAM MONITORING

SECTION X - DAM MONITORING

A. Purpose. A variety of instrumentation was needed to establish baseline control and provide ongoing monitoring related to groundwater levels and embankment movements. The need for this information was especiallty critical for this project because large scale modifications were being made to an operating dam.

B. Piezometers. Fourteen piezometers on the top of the dam were affected by construction activities (P-40, 44, 45, 50, 51, 56 and P-60 through 67). They were installed in the mid-1980's as part of the investigative phase for this contract. Most of these were modified when the dam was lowered from El.1250 to El.1240 during the construction phase. Eleven additional piezometers were installed under this contract; Seven in the dam core (P-101 through 107) and four in the narrow overburden spur between the canyon and the spillway (P-108 through 111). All new piezometers were 1 1/4-inch I.D., Schedule 80 P.V.C. pipe. Each 6-inch hole had 3-stages except P-111, and were numbered "1" (deep) through "3" (shallow). They were installed using a Schramm T-685H truck-mounted, air rotary drill using a 7-7/8inch tricone bit and the biodegradable drilling fluid, "Clearmud". The 4 piezometers in the spillway were drilled first, followed by the core piezometers.

The contractor had an extremely difficult time driving casing in the spillway spur and this problem continued with installation in the dam's core. As a result, the requirement to use temporary 6-inch casing to install the piezometers was waived. While the original condition of the core would have likely mandated its use, the recompression of the dam core made casing installation difficult and extraction in one piece all but impossible. All piezometer holes drilled in the core remained open and were flushed through the drill rods until return fluid cleared. The rods were pulled and the PVC tubes installed, backfilled and sealed to their target depths without problems. While fourteen of the existing piezometers were cut when the top of the dam was lowered, only nine were raised back up to the new top of dam (El. 1257). During the construction activities, five of them were rendered unsalvageable (P-44, 45, 50, 56 and 60).

The new piezometers were read on a weekly basis after installation for a period of about 6 to 8 weeks before the depths were sounded again. At that point, 5 to 30 ft. of sediment were found in the bottoms of most tubes with one (P-105-2) having 122 feet. With the exception of P-102-3, all piezometers were cleaned out with high pressure air varying from 1200 psi in the solid sections, to 500 psi in the 20 foot slotted ends. This operation was successful in removing smaller amounts of sand and silt in most of the tubes. P-103, P-105, and P-107 had denser plugs of cohesive and noncohesive fines (including particulate grout) in several of their tubes. This required several passes with the flushing equipment. Generally, the holes that were flushed twice showed the reformation of 3 to 5 ft. of sediment between operations (a day

apart). Readings obtained after cleaning suggest at least two inter-tube seals were ruptured by excessive pressures (P-105 and P-107).

It has been postulated that the sediment influx is a result of the new piezometers' placement in the recompression grouting zone. In this scenario water, bentonite and bentonite/grout mixtures present in the core prior to grouting were trapped and subjected to high compressive forces. Piezometers drilled in this zone then acted as relief drains, dissipating high pore pressures in the immediate area of the holes and in the process, moving sediments into the piezometer tubes. This high pore pressure is also reflected in the abnormally high core piezometric levels, which tends to "mask" pool-induced pressures in the core. It may take a while to fully dissipate these pressures, which were believed to have been further aggravated by raising the dam an additional seventeen feet.

C. Inclinometers. Seven inclinometers were installed during this contract for a total of 2,658 LF. Six are installed inside the concrete cutoff wall (Panels 123, 126, 130, 133, 137 and 139), while the remaining unit (Hole No. 199) is located 15 ft. downstream of Panel 133 (Plate 5). The contract originally provided for attachment of 5-inch steel pipes to the cutoff wall structural steel guide members, full depth. Inclinometer casings would then have been installed in the pipes after the panel concrete was placed. The structural steel guide members were deleted by the VECP, so inclinometers ended up being

installed in several concrete quality control/exploratory holes cored in the wall. The casing was pressure-grouted through one-way valves installed in the bottom of the casing.

The inclinometer casing is 2.75 in O.D. (2.32 in I.D.), non-corrodible plastic manufactured by SINCO of Seattle, WA. The casing is flush-coupled, self aligning pipe with two sets of casing groove spirals, one perpendicular to the other. One of the groove sets is aligned perpendicular to the cutoff wall centerline.

D. Settlement Monuments. Nine of the original dam settlement monuments (9A, 9B, 9C, 10 and 11 on upstream face; 2, 3, 4 and 7A on the downstream face) were incorporated into the contract dam monitoring program. Readings on these were taken in 1950 and then again in 1985, prior to construction. During this period, maximum settlements occurred through the top central portion of the upstream and downstream faces, dropping 1.3 ft. and .9 ft., respectively. These existing monuments were supplemented with 12 monuments on each of the upstream and downstream "shoulders" of the work platform at El. 1240, to monitor dam movements during construction.

Under the contract QC plan, the contractor installed offset points in the upstream and downstream concrete guide walls every 100 ft. for horizontal and vertical control of the cutoff wall panels. It was not until excavation of panel 143 in late July 1989, that a broad subsidence was noted along the guide

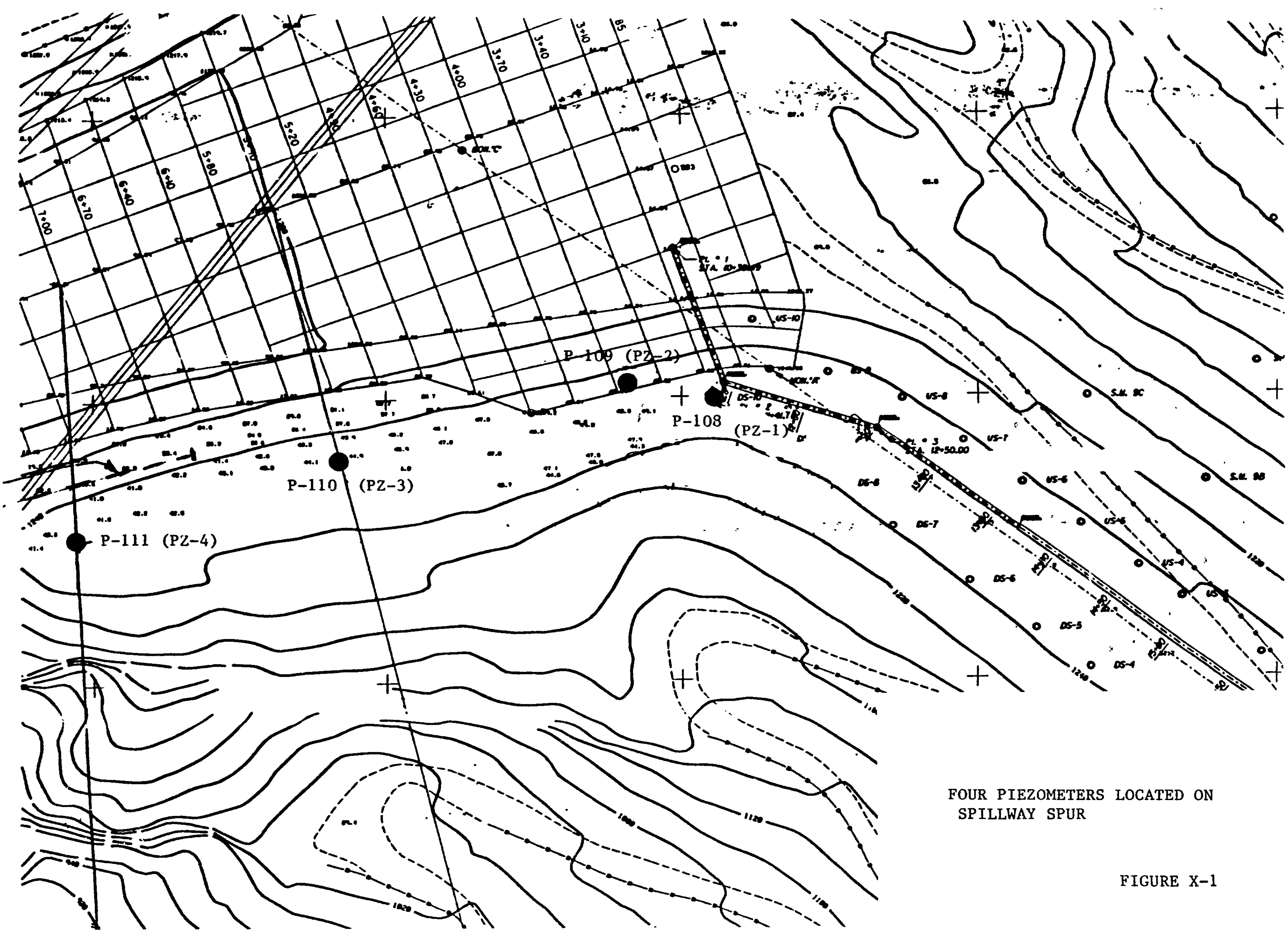
walls through the central portion of the dam. The initial readings indicated a maximum drop of .09 ft. in the deep section of the canyon. At that point supplemental monuments were added every 50 ft. between Station 13+00 and 17+00 to obtain localized settlement data. Detailed information on this settlement is presented in Section "IX".

E. Seismic Recorder. The two existing seismic recorders on the dam were removed by the U.S.G.S. prior to construction activities. The concrete housing for the recorder on the top of the dam was removed until the dam was raised at the end of the contract, then replaced, while the station at the downstream toe of the dam remained in place, but inactive.

PIEZOMETERS
(In Sequence of Installation)

Piezometer No.	Dates Drilled	Total Depth	No. Stages	C.O.W. Stationing
PZ-4 (P-111)	21-28 Mar 90	156	2	Spillway Spur (Fig. X-1)
PZ-3 (P-110)	29-31 Mar 90	200	3	" " " "
PZ-2 (P-109)	2-5 Apr 90	200	3	" " " "
PZ-1 (P-108)	6-10 Apr 90	200	3	" " " "
P-101	10-12 Apr 90	284	3	15 + 32.4 14.00 Rt.
P-102	12-13 Apr 90	364	3	15 + 10.0 15.00 Rt.
P-103	13-17 Apr 90	385	3	14 + 84.6 14.20 Rt.
P-106	17-23 Apr 90	290	3	15 + 07.9 11.60 Lt.
P-107	23-25 Apr 90	383	3	14 + 22.5 12.80 Lt.
P-105	25-30 Apr 90	400	3	14 + 24.9 15.50 Rt.
P-104	30 Apr-2 May 90	440	3	14 + 55.7 15.30 Rt.
		3,302		

TABLE X-1



FOUR PIEZOMETERS LOCATED ON
SPILLWAY SPUR

FIGURE X-1

SECTION XI

CONCRETE MATERIALS, BATCHING AND PLACING

SECTION XI - CONCRETE MATERIALS, BATCHING, AND PLACING

A. Aggregate Source

1. General. The aggregates for the cutoff wall and other incidental concrete work were supplied by the Cadman Sand and Gravel Company, 26111 S.E. Green Valley Road, Black Diamond, Washington; Washington State Pit No. A-455. The company at the beginning of this contract was named the Flintstone Concrete Company, Inc. The site is located approximately 1.5 miles south of the town of Black Diamond on State Route 169. The 117.25-acre site is situated in portions of the Northeast Quarter of the Northeast Quarter of Section 26, and the Northwest Quarter of Section 25, Township 21 North, Range 6 East, W.M., King County. The site extends from S.E. 352nd Street on the north to the Green River on the south. The easterly boundary is State Highway 169.

2. Site Description and Material Distribution. The site is underlain by glacial recessional outwash composed of well-sorted sand and pebble-cobble gravel deposited in an outwash plain. Materials are coarser at the surface, then become progressively finer with depth. A pervasive sand lens occurs in the upper portion of the sequence on both the eastern and western site boundaries, encircling the coarse granular materials. This sand is predominantly 100-mesh material, with between 4 and 12 percent minus-200 mesh silt and clay particles. Over the entire vertical sequence, there is good particle size distribution such that this represents a good crushing site as well as a good cement concrete

aggregate site.

3. Properties. The coarse aggregate is made up of 7% granite and other medium grained igneous rocks; 18% fine grained porphyritic rocks; 29% metavolcanics, flows, and breccia; 45% tuff, tuffaceous sandstone and siltstone; and 1% weathered, somewhat crumbly particles. Physically, the aggregate particles are hard, dense, and structually sound with a general subangular to subround particle shape and smooth surfaces. The aggregate particles are free of coatings. There are some rock particles with chalcedony, a potentially harmful form of quartz, and volcanic glass (although much of the glass is devitrified).

The natural sand is made up of 70% rock particles (igneous, volcanic, metamorphics); 3% siliceous and chert; 21% quartz; 3% feldspar; and 2% miscellaneous "heavy" minerals. Less than 0.5% were soft and/or highly weathered.

4. Supplemental Source. The Corliss Redi-Mix Company, 29410 Enumclaw-Chinook Pass Road (State Highway 410), Enumclaw, Washington; Washington State Pit No. A-44 was the originally submitted site by the contractor for the aggregate supply. The contractor reports that a lack of good quality control and an anticipated short supply of the proper aggregates at this source necessitated changing suppliers. The sand from the Corliss pit was used for the first month (Jun 5, 89 to Jul 12, 89) in the tremie concrete mix and was incorporated into the first 15 panels. This source was subjected to a petrographic examination in

1984 and quality tests in January 1989.

The site is situated in portions of the Southwest Quarter of Section 20, and the Northwest Quarter of Section 29, Township 20 North, Range 7 East, W.M., King County.

B. Aggregate Production

1. Aggregate Processing/Wash Plant. The pit run material was processed at the Black Diamond site with the subcontractor wash/crush processing facility. It was made up of two primary screening sections. The plant produced both crushed and naturally rounded aggregates. The oversize material went to the crushing segment of the plant and the desired aggregate was sorted via the arrangement of three decks of three screens each and sloped at 15-degrees. A 7/8-inch screen and two 3/4-inch screens constituted the first deck; a 3/8-inch screen and two 1/4-inch screens make up the second deck; and the third deck consists of 8-inch by 1-inch slots that run the length of the deck and parallel with the aggregate flow. The sand screw and the Lin-A-Tex separator are used in the sand production. A Kawasaki 95Z2 or 95Z3 front-end loader was used to excavate and transport the material to the plant.

2. Coarse Aggregate Production. The concrete aggregates are produced on the second deck of screens. These aggregates are then passed to a gravel washer where the auger agitates the gravel and the water floats the undesirable products

out. Pea gravel is scalped off on the third deck. Whatever passes all three screens goes to the sand deck for the sand size production.

3. Fine Aggregate Production. From the sand deck, the material proceeds through a chute to the sand screw where the silt and organics are washed out. The sand is dewatered as it progresses up the screw. The final processing for the sand occurs in the Lin-A-Tex separator. This separator recycles the wash water and reclaims the larger sand particles by stirring the particles into suspension and using a vacuum regulated to remove the excess deleterious fines. The reclaimed sand is blended back into the already processed sand.

4. Grading and Quality Requirements. The coarse aggregates produced for the tremie concrete had a nominal maximum size (MSA) of 3/4-inch and was natural (rounded) with not more than 10 percent of the particles having fractured faces. The limits for deleterious substances and physical property requirements conformed to ASTM C33, Class 4S. The gradation envelope size number was a ASTM C33, No. 67 (3/4 in. to No. 4) with the following weight percent passing amounts:
FIGURES/TABLES IN APPENDIX

Sieve Size	% passing
1-inch	100
3/4-inch	90 - 100
1/2-inch	-
3/8-inch	20 - 55
No. 4	0 - 10
No. 8	0 - 5

The nominal maximum size (MSA) for the structural concrete was 7/8-inch. The limits for deleterious substances and physical property requirements conformed to ASTM C33, Class 4S. The gradation envelope size number was a ASTM C33, No. 57 (1-in. to No. 4) with the following weight percent passing amounts:

Sieve Size	% passing
1 1/2-inch	100
1-inch	95 - 100
3/4-inch	-
1/2-inch	25 - 60
3/8-inch	-
No. 4	0 - 10
No. 8	0 - 5

The gradation and quality requirements for all of the fine aggregate produced conformed to ASTM C33. The following table lists the weight percent passing amounts:

Sieve Size	% passing
3/8-inch	100
No. 4	95 - 100
No. 8	80 - 100
No. 16	50 - 85
No. 30	25 - 60
No. 50	10 - 30
No. 100	2 - 10
No. 200	0 - 5

5. Storage. The aggregates were stockpiled at the Black Diamond site until needed for concrete placements at Mud Mountain Dam. The material was trucked 14 miles to the batch plant at the construction site. A variety of truck types were

used to transport the aggregates. Generally, 24-25 tons were hauled by each truck and trailer combination.

6. Amount Produced. The total amount of fine and coarse aggregate produced for the tremie and other concrete placed under this contract was approximately 32,210 tons.

7. Water Supply. Water for processing the aggregate was obtained from a spring located on the Cadman property.

C. Cementitious Materials.

1. Portland Cement. The specifications allowed the use of an ASTM C150 low alkali Type II Portland cement with a maximum amount of tricalcium aluminate of 8 percent. The false set requirements were specified also. The Ideal Basic Industries, Cement Division, Seattle, Washington supplied all of the cement for the concrete placed under this contract. The cement was trucked to the site in increments of 60,000 to 70,000 pounds and transferred to either the 115-ton cement silo or the 175-ton guppy.

2. Pozzolan Source. Pozzolan meeting the requirements of ASTM C618 Class F, with the optional requirements of Table 1A for magnesium oxide and Table 2A was selected for use by the contractor. All of the pozzolan was supplied by Pozzolanic Northwest, Mercer Island, Washington. It was delivered to the site

in increments of 50,000 to 70,000 pounds and placed in the 100-ton fly ash silo.

Only the tremie concrete used pozzolan as a cement replacement. The mix was designed with 30 percent of the total absolute volume of the cementitious materials using pozzolan. Approximately 3,843 tons of cement and 1,133 tons of pozzolan were used on this project.

D. Admixtures

1. Air-entraining Agent. The air-entraining admixture selected by the contractor was MB-VR Standard produced by Master Builders, Inc., Cleveland, Ohio. This aqueous vinsol resin solution has been neutralized with sodium hydroxide. The ratio of sodium hydroxide to vinsol resin is 1:6 with a nominal 13 percent by weight residue when dried at 105-degrees C. MB-VR Standard meets the requirements of ASTM C260 and CRD-C13-86.

2. Water-reducing Admixture. Master Builders, Inc. supplied the water-reducing admixture MBL-82. It meets the requirements for a Type A, water-reducing, Type B, retarding, and Type D, water-reducing and retarding admixture as specified by ASTM C494 and CRD-C87. This admixture was used in the incidental concrete mixes such as the spillway replacement and cutoff wall extension.

3. High Range Water-reducing (Super-plasticiser) and Retarding Admixture. The chosen admixture for use in the tremie concrete mix was Master Builders

Rheobuild 561. This agent is a Type G, high range water-reducing and retarding admixture and it complies with the requirements of ASTM C494 and CRD-C87.

4. Coloring Compounds. For verticality/continuity checks of the primary panel concrete, a red or black dye will be used. The manufacturer of these dyes is the Davis Colors Company, Los Angeles, California. The red color is Davis Colors No. 1100, iron oxide red and the black is Davis Colors No. 8084, Supra instant black, (carbon black). The amount used in the concrete is the ratio of 1 pound per 100 pounds of cementitious material.

E. Concrete

1. Batch Plant. The batch plant was erected on-site by the subcontractor Cadman Sand and Gravel. It was a CON-E-CO Lo-Pro dry batch plant. Plant fabrication started on March 28, 1989 and it was operational by May 1989. The plant capacities are as follows:

THEORETICAL PRODUCTION CAPACITY - 120 to 150 cubic yards per hour

BATCHING CAPACITY - 10 cubic yards

STORAGE BIN CAPACITIES - Aggregate: 98 cubic yards heaped
(two compartments with eight gates)

Cement Silo: 1170 cubic yards (115 tons)
Pozzolan Silo: 1425 cubic yards (100 tons)
Additional Storage Guppy: 1781 cy (175 tons)
Admixture: AEA - 500 gallons
WRA/HRWRA - 2000 gallons

The 30-inch wide loading conveyor could load the aggregate storage hoppers at a rate of 630 tons per hour. The equally wide aggregate batch conveyor could deliver the aggregates to the truck mixer at a pace of 600 tons per hour.

Cement and flyash were weighed in the forward hopper with the cement being weighed prior to the flyash. The aggregates were weighed in the aft hopper. The scales were of the suspension hopper type with dials. These dials were fitted with potentiometers to provide electrical impulse for the computer console. The weighing accuracy was checked monthly while the batching accuracy was monitored weekly.

All of the batching operations were automatically controlled by the computer console without interruption. This system "learned" with each subsequent batch and was able to accurately control the batching sequences. The concrete mixes were initially made according to the following order as the transit mixer is rotating at mixing speed:

- Load approximately 85% of the water with the air-entraining agent.
- Load approximately 50% of the aggregates.
- Loading the rest of the aggregates continuously with the cementitious materials.
- Loading the remainder of the water with the super plasticiser.
- Rotate at mixing speed for 70 revolutions.

2. Winterization of Operations. Since the concreting operation went into the winter months, certain precautions were taken to winterize the batch plant and materials:

- The coarse and fine aggregate stockpiles were placed on 10 inch steel pipes that spanned the length of the stockpile and had forced-air propane heaters installed to heat the aggregates.
- The aggregate bins were enclosed with plywood to retain heat.
- The admixture containers and dispensers were enclosed and equipped with heat tape.
- The aggregate bins were roofed to keep precipatation from collecting in them.
- The water tanks were heated with a propane burner.
- While the dam core protected most of the concrete, the top was covered with an insulated blanket after the excess concrete was removed.

3. Transit Mixers. The transit mixers used on-site for mixing and transporting concrete were either Rex 470 types or Rex 770 types, (the one having chain-driven drums and the other hydraulic-driven drums). The mixing capacities were either 8 cy or 10 cy. An 8 cy maximum limit for each truck was used due to the high slump of the concrete and the tendency to spill out of the truck when ascending the ramp to the work platform. One truck was randomly selected for the mixer uniformity test, initially, and every three months of concrete placing. These trucks were also inspected for blade condition, concrete accumulation on the blades and the charge and discharge chutes, operable revolution counters, graduated water gauges, and that the water discharge pipe points into the center of the drum.

4. Tremie Operation. The tremie pipe diameter was 10-inches. All connections in the pipe were watertight. Each tremie pipe was equipped with a funnel-shaped hopper of approximately 1.1 cy capacity. The hopper had a screen half way covering the opening to prevent the passing of lumps of concrete larger than three inches in diameter. Tremie pipes were at a maximum of 15-feet apart. One to two tremie pipes were used. A service crane was available for raising and lowering the tremie pipe.

A portable work platform stradled the trench opening and rested on the guide walls that were constructed prior to the Hydrofraise excavations. "Dog ear" support plates that were hinged to open to pass the upward moving tremie pipe held the tremie pipe sections at the coupling to allow pipe sections to be added or removed. The tremie pipe when at the longest extension into the panel trench (400+ ft) was held vertically with little, if any, lateral movement. Steel shoulder plates were inserted in the guide wall to form the top few feet of the panel because of the pre-trenching on either side of the panel trench.

The pipes were lowered to within one to two feet of the trench bottom. A wood sphere or vermiculite "go-devil" was used in each tremie. Two to four transit mixers were batched for the start of a placement. The first load of concrete was delivered as quickly as possible to push the go-devil out of the pipe and to create a seal around the tremie pipe by immersing it in the concrete.

Immersion of the pipe was maintained between 10-feet and 30-feet. Delivery was such as to produce the flattest concrete surface that can practically be achieved. Concrete placement continued without interruption until the concrete reached the top of the guide walls. A minimum of 3 cy of concrete was usually allowed to overflow the trench to remove contaminated concrete and muck.

The concrete level was measured by sounding at frequent intervals so that the actual volume of concrete placed could be compared with the theoretical volume. The results were used to produce a concrete volume curve for each panel. The soundings also indicated when tremie cuts (shortening of the pipe length) were to occur.

The displaced slurry was pumped out of the trench to a holding pond for reclamation.

5. Go-devil. A retrievable, snug-fitting, non-collapsible, traveling go-devil was specified for use in the tremie pipe. The contractor wanted to use a 1-foot thick wad of vermiculite. Soletanche stated that the vermiculite go-devil was successfully used in previous tremie concrete placements. The Government was not convinced that it was retrievable. The plan decided upon was a combination of a 1-foot thick wad of vermiculite plus the wood sphere in each pipe as a go-devil.

The chance of recovery of eicher element of the go-devil was best in the

first minutes of placement than at any other time. It appeared that during the first few loads the go-devil usually returned. It was noticed that both the vermiculite and the spheres were seen on the surface, but the spheres were more easily recognized than the vermiculite in the slurry. The spheres were not always retrieved nor was the vermiculite always seen. There were approximately eleven spheres lost.

The second best opportunity for sphere recovery was toward the end of the placement. A search of the concrete and muck that was piled at the top of the panel occasionally provided the sphere that did not surface earlier.

The greater depths of the panels in the canyon inhibited the return of any of the go-devils.

6. Concrete Production. The specifications called for a batch plant capable of producing concrete at a rate of 100 cubic yards per hour for the tremie concrete and 50 cubic yards per hour for structural concrete. This plant satisfied both requirements. The 100 cy/hr was necessary due to the large amounts of concrete to be placed in the huge trenches. Some placements were expected to last for 24 hours and a heterogeneous mixing of plastic and set concrete was not a desired condition in these monolithic panels.

The first cutoff wall concrete placed was on June 5, 1989 and the concreting of the panels continued until it was interrupted by the recompaction

of the dam core by grouting, (Aug. 24, 89). Concreting resumed on December 8, 1989 and was completed on April 12, 1990.

A 100 cy/hr placement was never attained. Usually, the batch plant during the first hour or two could reach a 96 cy/hr batching rate but the placing of concrete set the pace. An overall average of 53 cy/hr was maintained during the cutoff wall placements with a high of 77 cy/hr achieved on February 16, 1990. The average was 47 cy/hr for, primarily, the Type I panels and 63 cy/hr for the Type II panels. The increase in placing speed for the Type II wall was due to changing the panel configuration from a three-bite panel which would have two tremie pipes to a one-bite panel with two tremie pipes. This change provided more concrete faster to a smaller trench. Also, the time for the expected long 24-hour placements was cut signficantly.

The total amount placed in the tremied cutoff wall was 18,277 cubic yards.

Concrete quantities were an overall average of 16 percent over the neatline estimate. The Type I walls, left and right banks, were 26 and 25 percent overrun, respectively. The Type II wall experienced the tightest control on overrun at 11 percent while this section had the greatest amount of concrete at 11,865 cubic yards. The recompaction grouting in the canyon area of the dam may have allowed the excavation of better shaped trenches and narrowed the gap between the theoretical and actual amounts.

The deepest panels required about 500 cubic yards of concrete. The panel that holds the world depth record (Panel 129 - 402.55 feet) was concreted on February 1, 1990 and contained 516 cubic yards. The largest concrete placement occured on January 16, 1990 for panel 127 with a capacity of 524 cubic yards.

7. Specified Concrete Properties. Three mix designs were approved for use for the work under this scope of work.

COMPRESSIVE STRENGTH

3,000 psi at 28-days for the tremie concrete.
3,000 psi at 28-days for all other concrete.
4,000 psi at 28-days for the spillway concrete.

SLUMP

7 to 9-inches for the tremie concrete.
2 to 4-inches for walls.
3-inches maximum for all other concrete.

AIR CONTENT

No requirement for entrained air in the tremie concrete.
5 to 7-percent for all other concrete.

WATER-CEMENT RATIO

0.45 for the spillway and exposed concrete.
0.50 for the tremie concrete (equivalent cement).

REQUIRED AMOUNT OF CEMENT

500 -pounds minimum for the tremie concrete.

TREMIE CONCRETE DESIGN PROPORTIONS

Materials	Original Mix	Adjusted Mix
3/4-inch, lbs.	1807.0	1790.0
Sand, lbs.	1421.0	1400.0
Cement, lbs.	383.0	401.0
Pozzolan, lbs.	117.0	124.0
Water, lbs.	257.1	257.1
HRWRA, fl. oz.	90.0	95.0
AEA, fl. oz.	2.3	4.4
	--------	--------
Total lbs.	3985.1	3972.1
Equivalent W/C	0.47	0.45
Air, %	3.0	5.3
Yield, ft3	27.0	27.0
Unit Wt., lbs/ft3	147.6	147.1

OTHER CONCRETE DESIGN PROPORTIONS

Materials, SSD	#606 - Other Concrete	#607 - Spillway Concrete
7/8-inch, lbs.	1960.0	1960.0
Sand, lbs.	1390.0	1230.0
Cement, lbs.	470.0	540.0
Water, lbs.	260.0	243.0
WRA, fl. oz.	28.0	42.0
AEA, fl. oz.	3.0	3.5
	--------	--------
Total lbs.	4080.0	3973.0
W/C	0.55	0.45
Air, % (+/-1)	6.0	6.0
Yield, ft3	27.0	27.0
Unit Wt., lbs/ft3	151.1	147.1

8. Concrete Coloring Sequence. A dye was added to the concrete in the Type II wall canyon section. The bottom 20 feet of the primary panels was targeted for coloring. Some of the coloring was noted on the surface at the end of each placement but it is believed that most of the colored concrete remained

at the bottom of the panel. Coring of these panels verified that the bottom of the panels retained the colored concrete. Red and black were used, alternately. The requirement for the dye as a redundant check is discussed elsewhere in this report.

PANEL NO.	COLOR
15	Black
17	Red
119	Red
121	Black
123	Red
125	Black
127	Red
129	Black
131	Red
133	Black
135	Red
137	Black
139	Black
141	Black

9. Concrete Quality. The coring of the concrete panels showed the concrete to be of an excellent quality. The overall compressive strength average at 28-days was 5,240 psi and well over the required minimum of 3,000 psi.

A few problem areas (panels 7, 15, and 17) exhibited anomalies that were of a cosmetic nature only. Cores from these panels revealed honey-combed and washed-out or gravely zones, yet the compressive strengths from the core samples ranged from 4510 psi to 8990 psi. Two of the sample cores were taken from areas of visibly solid concrete; the four other samples came from areas of light to severe washout of the mortar component.

The small amount of water loss recorded during the down-hole packer tests in these panels were, evidently, the result of poor testing methods and equipment. The drill holes were grouted with no grout takes observed.

10. Concrete Cohesiveness. The concrete for tremie placement needs to be fluid enough to easily flow down through the tremie pipe. The ratio of the binding phase (mortar) should be large enough to avoid any excessive friction between the coarse aggregates and it must be cohesive and viscous enough to avoid bleeding and segregation of the concrete during and after placement. These conditions are usually fulfilled when the binding phase is rich in fine elements (cement, pozzolan, sand, and air).

The originally submitted mix design was used for the first eight concrete placements. It was during these placements, and especially for panels 15 and 17, that it was noticed in the tremie hopper, occasionally, that the concrete was not very cohesive. There were concrete slumps in excess of the 9-inch maximum limit, also. Concrete cores from panels 15 and 17 exhibited honey-combing and vertical channels.

The channels were likely the result of the upward displacement of water along with fine particles caused by the difference in specific gravities between the binding phase and the aggregates when the binding phase is not cohesive enough. It may be possible that some slight localized displacements of the ground under the hydrostatic pressure of the fluid concrete (at least twice as

dense as the slurry) had taken place, creating a path through which the bleed water and cement particles could have escaped and leaving the honey-combed effect in the concrete.

The contractor upon evaluating the mix design, cores, and the QC testing made some modifications to the concrete mix starting on June 27, 1989. The changes to the mix were a 25-pound increase in cementitious materials and a doubling of the air-entraining agent. This increased the ultimate compressive strengths slightly, which were already approaching the 5,000 psi area at 28-days. (The high concrete strengths are a by-product of the high amounts of cementitious materials used to achieve cohesiveness). The most effective contribution in avoiding bleeding and segregation was the consequent effort to maintain the concrete slumps in the specified range of 7 to 9-inches. (The optimum concrete slumps occured in the 8.25 to 9-inch range. The concrete flowed much more readily down the pipe and accelerated the placing of concrete at this consistency).

11. Slump Variability and Cement-balls. Slump variability was a problem of differing severity during all of the concrete operations of the cutoff wall. A number of factors contributed to this situation:

a. The minus 200 fraction of the sand in the beginning of the concrete operations was bordering the 5 percent limit and exceeded it on numerous occasions. The minus 200 percentage was roughly halved when the Corliss sand

was substituted with the Cadman pit sand.

b. The coarse aggregate stockpile was drying out in the hot summer weather and it was absorbing some of the batch water. A better sprinkling system was obtained by the contractor to maintain the coarse aggregate at SSD and minimizing the effect it has on the concrete.

c. The sand was used shortly after processing and delivery to the site. This material was holding an excess amount of water and wreaking havoc with the concrete slump. This problem was remedied by allowing the sand stockpile to drain for at least 24 hours before being used to achieve a constant condition in the sand.

d. Transit mixers would occasionally back up at the placement and the sun shining on the drum was effecting the slump. The causes of the back-ups were things like tremie pipe length shortening operations, concrete backing up in the hopper (usually signaling the need for shortening the tremie pipe), slurry pump and other equipment breakdowns, the slowing of the concrete placement as the concrete gets higher in the trench and less of a drop for concrete in the pipe, concurrent construction activities on the tight space of the dam work platform, and the sounding device stuck in the trench, etc. Coordination between the batch plant operator and the placing foreman by radio and a traffic flow plan helped to solve this problem.

e. The lack of homogeneity of the concrete in the transit mixers was troublesome. The concrete at the front of the drum (first out) was wetter than the concrete at the back of the drum. So, concrete slumps that were near the lower tolerance as first delivered would exceed the slump range on the lower end as the concrete delivery continued. The concrete would then be rejected. In order to improve the uniformity of the concrete in each load, the batching and mixing process was modified:

- The loading of 90% of the water versus 85% at first.
- Continuing the 70 mixing revolutions at the batch plant but adding an additional 20 mixing revolutions after transporting to the dam work platform and just before placement. This also minimized any segregation that might have occured during transit.

These changes produced a more consistent concrete and helped to speed the concreting operations. The incidence of cement-balls was greatly reduced although not totally eliminated. There was a break-even point in the batching operation. Cement-balls could be excluded at the expense of batching time but that would cause a problem with concrete delivery at the placement. Any cement-balls that continued to show up at the tremie hopper, would be broken apart by a laborer that directed the concrete delivery from the truck. Cores from the completed panels never showed any problem in the concrete from cement-balls.

Occasionally, a mixer contained concrete with a slump lower than specified.

Too stiff concrete was allowed to be redosed with the high range water-reducing admixture (superplasticizer) to bring the concrete to the desired slump. Redosed concrete was tested for compressive strength with the results being no different than the regular concrete.

12. Panel 37. Two attempts were made at concreting panel 37 with it being completed successfully on July 24, 1989. The first attempt at placing concrete on July 19, 1989 was fraught with problems. The contractor was placing concrete that appeared to be in the 4-inch to 5-inch range and it would not go down the tremie without assistance. The tremie was being raised and lowered to facilitate the concrete going down the pipe. After 4-5 times of pumping the tremie up and down, the concrete started to flow down the pipe, but the hopper would not settle back down onto the holding platform stretched across the open trench from which it was initially raised. The tremie pipe appeared to be hitting something solid at approximately 18-inches above the initial depth of the pipe. Several more tries were made to seat the hopper but no progress was being made and the tremie looked to be bouncing on something and making a solid sounding noise on contact. The foreman instructed the crew to cut a 3-feet section of pipe from the tremie length and continue the concrete operation. The contractor was notified in writing that the Government intended to core the concrete at this location and if substandard concrete is discovered the contractor will have to remove the panel at no cost to the Government. The contractor decided to scrub the operation and immediately began to remove the 176 cy of concrete (out of a trench that would eventually hold 400 cy), via the

clam-shell bucket.

A Government inspector reported on July 21, 1989 that he examined the hydrofraise picks removed from the cutting wheels that were used at the same depth that the problem occured and notes that they appeared in poor condition from excavation on rock. It may be that a large boulder fell from the side of the trench walls, during or before concreting, and prevented the tremie pipe from being lowered to the original position. Panel 37 is located between panels 35 and 39 where extensive cracking of the dam occured which may have induced some instability of the trench walls.

13. Bentonite Intrusion into Concrete. The contractor was granted a waiver on the maximum tremie embedment constraint for the concrete placement of panel 141 on February 22, 1990. This was due to the bentonite slurry intrusion discovered in the concrete of panel 137 at the approximate locations of the tremie cuts. That panel was concreted on December 19, 1989. The contractor maintained that this was due to too little embedment of the tremie and the turbulence that occurs at the greater depths in the panel. The change was allowed in anticipation of remedying the problem.

The deeper embedment of the pipe caused the concrete to exit the pipe much slower and the tremie had to be jacked up and down throughout the placement instead of toward the finish. The tremie embedment was as much as 100 feet and when tremie sections had to be removed the time was increased four fold.

(Approximately from 5 minutes to 20 minutes and required coordination with the batch plant to keep from backing-up transit mixers loaded with concrete).

On March 13, 1990, the contractor modified the embedment depth and tremie cut operation. There would be a minimum embedment of 30 feet and two sections (43.3 feet) cut at a time until about 45 feet from the top of the panel. This proved to be much more efficient and the placing rate was 72 cubic yards an hour for this placement.

The bentonite slurry intrusion was not observed in any panels cored since the waiver was granted on February 21, 1990.

14. Spillway Replacement Concrete. The spillway wall on the left side had a section removed to an elevation of 1240 feet when Soletanche prepared the work platform for the cutoff wall construction. A slot was also cut into the spillway wall and floor as the cutoff wall would jut into the spillway about 56 feet from the top of the spillway wall.

The specified cutoff wall tremie concrete was placed to the spillway surface and a lean mix concrete deposited on top. When the ramp to the work platform was removed, the weaker concrete was broken out and the tremie concrete was excavated to a minimum depth of 2 feet.

The replacement concrete was specified at 4,000 psi in 28-days with it

averaging at 4,910 psi and requiring 227 cubic yards to complete.

15. Cast-in-Place Cutoff Wall Extension. The tremie concrete cutoff wall was constructed to the elevation of 1240 feet which was the elevation of the work platform constructed by Soletanche. The contract called for an additional 13 feet of one foot thick wall to the elevation of 1253 feet. The wall extension would be a structural cast-in-place concrete wall.

Prior to proceeding with the construction of the wall extension, the top of the tremie concrete wall between the guide walls was excavated with one set of the Hydrofraise cutter wheels. The cutter wheels were mounted on the upper plate of a Caterpillar D-9 and the power pack was carried along side in an end-dump. This arrangement of cutting wheels and tractor was nicknamed "the American fraise."

The concrete was trimmed to anywhere between 0.5 foot to 3 feet, depending on the quality of the top concrete. Special care was given to the joints between the panels. The depth of excavated concrete was deeper at the joints.

An air/water jet was used to clean the excavated surface, a bond-coat applied, and a leveling course of 3,000 psi concrete was placed in this excavated section to within 6-inches of the top of the guide wall. Two rows of dowels were grouted into the drilled holes in the leveling course. The concrete surface was sand-blasted and cleaned, the reinforcement mats attached, a

bond-coat applied, the formwork raised into place before the concrete was deposited by with pump truck.

The wall extension was constructed in sections and consisted of 38 panels. Alternate panels were concreted. The panel joints were sealed with a waterstop. Four sets of forms were available with one usually serving as a back-up. The concrete work was accomplished by Kodo Construction Co. which was a subcontractor to subcontractor KSC Inc.

The cutoff wall extension was constructed between May 22, 1990 and August 7, 1990 and required a total of 402.5 cubic yards.

The concrete compressive strength was specified at 3,000 psi in 28-days. Compressive strength averaged 5,030 in 28-days.

16. <u>Cascade Creek Diversion</u>. The Lower Cascade Creek flows down a steep slope onto the top of the dam at the southeast end. A considerable amount of debris ravels down to the dam. A diversion basin was constructed to collect the debris for easy removal and to divert the creek to the upstream side of the dam.

The concrete was cast-in-place for the headwall section with the remainder of the concrete was screeded and hand-floated. A total of 39.5 cubic yards was placed for this structure.

The concrete compressive strength was specified at 3,000 psi in 28-days. Compressive strength averaged 4,460 in 28-days.

PLACING RATES FOR MUD MOUNTAIN DAM CUTOFF WALL TREMIE CONCRETE

DATE	HRS TO COMPLETE Placement	HRS TO COMPLETE Cumulative		CY CONCRETE PLACED Placement	CY CONCRETE PLACED Cumulative		CY CONCRETE ANTICIPATED Placement	CY CONCRETE ANTICIPATED Cumulative		CY/HR* Pl.	CY/HR* Cum		TREMIES
05-Jun-89	5.7	5.7		208	208		176	176		37	37		2
07-Jun-89	7.3	13.0		440	648		323	499		60	49		2
08-Jun-89	5.6	18.6		240	888		200	699		43	47		2
09-Jun-89	4.3	22.9		195	1083		167	866		46	46		2
16-Jun-89	9.5	32.4		517	1600		487	1353		55	48		2
19-Jun-89	2.0	34.4		96	1696		79	1432		48	48		1
20-Jun-89	4.0	38.4		229	1925		185	1617		57	49		2
22-Jun-89	7.8	46.2		306	2231		258	1875		40	48		2
27-Jun-89	4.8	51.0		240	2471		192	2067		51	48		2
28-Jun-89	4.3	55.3		231	2702		187	2254		54	49		2
06-Jul-89	5.7	61.0		91	2793		70	2324		16	46		1
07-Jul-89	4.7	65.7		351	3144		285	2609		75	48		2
10-Jul-89	5.0	70.7		204	3348		176	2785		41	48		1
11-Jul-89	2.9	73.6		112	3460		87	2872		39	47		1
12-Jul-89	5.3	78.9		202	3662		156	3028		38	47		1
13-Jul-89	4.7	83.6		336	3998		265	3293		72	48		2
21-Jul-89	5.7	89.3		147	4145		121	3414		26	47		1
24-Jul-89	6.3	95.6		400	4545		329	3743		63	48		2
26-Jul-89	2.2	97.8		102	4647		73	3816		46	48		1
26-Jul-89	4.4	102.2		248	4895		152	3968		56	48		2
28-Jul-89	2.1	104.3		64	4959		36	4004		30	47		2
28-Jul-89	4.3	108.6		174	5133		135	4139		40	47		2
03-Aug-89	4.6	113.2		264	5397		221	4360		57	47		1
04-Aug-89	2.7	115.9		104	5501		73	4433		39	47		1
07-Aug-89	3.5	119.4		212	5713		170	4603		61	48		2 *
08-Aug-89	2.6	122.0		139	5852		115	4718		54	48		1
08-Aug-89	3.2	125.2		161	6013		131	4849		50	48		1
09-Aug-89	2.5	127.7		136	6149		122	4971		54	48		2 *
10-Aug-89	3.5	131.2		136	6285		94	5065		39	48		2
11-Aug-89	2.6	133.8		176	6461		148	5213		68	48		1
14-Aug-89	2.2	136.0		160	6621		148	5361		73	49		2 *
15-Aug-89	3.0	139.0		128	6749		103	5464		43	49		1
15-Aug-89	2.2	141.2		92	6841		74	5538		42	49		1
16-Aug-89	1.9	143.1		92	6933		74	5612		48	49		1
17-Aug-89	2.4	145.5		100	7033		74	5686		42	49		1
17-Aug-89	2.2	147.7		84	7117		73	5759		38	48		1
18-Aug-89	1.8	149.5		60	7177		50	5809		33	48		1
18-Aug-89	2.2	151.7	After	80	7257	After	62	5871	After	36	48		1
23-Aug-89	1.5	153.2	Grouting	64	7321	Grouting	50	5921	Grouting	43	47		1 *
23-Aug-89	1.7	154.9	Dam Core	76	7397	Dam Core	67	5988	Dam Core	45	47	P/G	1 *
24-Aug-89	4.6	159.5	Cumulative	232	7629	Cumulative	205	6193	Cumulative	50	47	Cum	1
08-Dec-89	6.6	166.1	6.6	390	8019	390	349	6542	349	59	48	59	2 +
12-Dec-89	7.0	173.1	13.6	488	8507	878	451	6993	800	70	48	65	2 +
19-Dec-89	7.9	181.0	21.5	486	8993	1364	448	7441	1248	62	49	64	2 +
22-Dec-89	5.0	186.0	26.5	356	9349	1720	319	7760	1567	71	49	65	2 +
10-Jan-90	5.4	191.3	31.9	246	9595	1966	213	7973	1780	46	49	62	2 +
16-Jan-90	9.1	200.4	41.0	524	10119	2490	454	8427	2234	58	49	61	2 +
18-Jan-90	5.4	205.8	46.4	332	10451	2822	287	8714	2521	62	49	61	2 +
23-Jan-90	7.8	213.6	54.2	496	10947	3318	447	9161	2968	64	50	61	2 +

TABLE XI-1(a)

PLACING RATES FOR MUD MOUNTAIN DAM CUTOFF WALL TREMIE CONCRETE

PANEL	DATE	HRS TO COMPLETE			CY CONCRETE PLACED			CY CONCRETE ANTICIPATED			CY/HR*		
		Placement	Cumulative	P/G Cum.	Placement	Cumulative	P/G Cum.	Placement	Cumulative	P/G Cum.	Pl.	Cum	P/G
124	29-Jan-90	5.8	219.4	60.0	361	11308	3679	331	9492	3299	62	50	61
118	30-Jan-90	3.9	223.3	63.9	230	11538	3909	205	9697	3504	59	50	61
129	01-Feb-90	7.1	230.4	70.9	516	12054	4425	457	10154	3961	73	51	62
139	06-Feb-90	6.4	236.8	77.3	468	12522	4893	430	10584	4391	73	51	63
122	12-Feb-90	5.5	242.3	82.8	336	12858	5229	302	10886	4693	61	51	63
126	16-Feb-90	5.4	247.6	88.2	412	13270	5641	370	11256	5063	77	52	64
141	22-Feb-90	6.7	254.4	94.9	423	13693	6064	384	11640	5447	63	52	64
120	05-Mar-90	5.5	259.8	100.4	312	14005	6376	280	11920	5727	57	52	63
135	07-Mar-90	7.4	267.2	107.8	499	14504	6875	453	12373	6180	67	52	64
130	13-Mar-90	7.0	274.2	114.8	502	15006	7377	454	12827	6634	72	53	64
140	20-Mar-90	6.9	281.1	121.7	460	15466	7837	409	13236	7043	67	53	64
134	26-Mar-90	8.3	289.5	130.0	502	15968	8339	453	13689	7496	60	53	64
128	27-Mar-90	8.1	297.5	138.1	503	16471	8842	455	14144	7951	62	53	64
138	30-Mar-90	8.6	306.1	146.7	494	16965	9336	443	14587	8394	57	53	64
132	02-Apr-90	8.5	314.6	155.2	500	17465	9836	450	15037	8844	59	53	63
136	06-Apr-90	8.7	323.3	163.9	504	17969	10340	454	15491	9298	58	53	63
142	12-Apr-90	5.1	328.4	169.0	308	18277	10648	258	15749	9556	60	53	63
		328.4 TOTAL			18277 TOTAL			15749 TOTAL			53 AVG		

P/G - After Grouting Dam Core (Post Grout)

+ Two tremies in single bite trench.

* Earlier placement hours have better placing rates. The concrete operation slows as the concrete reaches the top of the trench. These numbers reflect the overall process, from the 1st concrete placed through the last concrete placed. Cumulative in this column is the accumulated average to the point indicated.

Spillway section. The bottom of the trench was 80.05 feet from the top of the ramp built across the spillway surface. The cutoff wall concrete was placed only to the spillway surface.

TABLE XI-1(b)

Mud Mountain Dam Seepage Control Cutoff Wall Dimensions and Theoretical/Actual Concrete Amounts

PANEL). / BITE	STATION LEFT	STATION RIGHT	PANEL LENGTH,ft	PANEL DEPTH,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUNT CONC. PLACED, yd3	AMOUNT OVER AMOUNT, yd3
1			13.50					18		
A	1847.00	1842.70		10.01	2.67	4.30	4.26			
B	1842.70	1833.50		15.09	2.67	9.20	13.73			
2	1833.50	1825.10	8.40	20.01	2.67	9.20	18.20	18	64 total 1&2	28
3			22.80					94	136	42
A	1825.10	1815.90		33.07	2.67	9.20	30.09			
B	1815.90	1811.50		44.72	2.67	4.40	19.46			
C	1811.50	1802.30		49.31	2.67	9.20	44.86			
4	1802.30	1793.90	8.40	54.46	2.67	9.20	49.55	50	60	10
5			22.80					135	174	39
A	1793.90	1784.70		56.43	2.67	9.20	51.34			
B	1784.70	1780.30		57.41	2.67	4.40	24.98			
C	1780.30	1771.10		63.98	2.67	9.20	58.21			
6	1771.10	1762.70	8.40	68.40	2.67	9.20	62.23	62	80	18
7			22.80					167	195	28
A	1762.70	1753.50		71.00	2.67	9.20	64.59			
B	1753.50	1749.10		73.49	2.67	4.40	31.98			
C	1749.10	1739.90		77.10	2.67	9.20	70.14			
8	1739.90	1731.50	8.40	76.57	2.67	9.20	69.66	70	91	21
9			22.80					176	208	32
A	1731.50	1722.30		77.10	2.67	9.20	70.14			
B	1722.30	1717.90		78.41	2.67	4.40	34.12			
C	1717.90	1708.70		79.40	2.67	9.20	72.24			
10	1708.70	1700.30	8.40	86.71	2.67	9.20	78.89	79	96	17
11			22.80					200	240	40
A	1700.30	1691.10		86.94	2.67	9.20	79.10			
B	1691.10	1686.70		89.73	2.67	4.40	39.04			
C	1686.70	1677.50		90.39	2.67	9.20	82.23			
12	1677.50	1669.10	8.40	95.96	2.67	9.20	87.30	87	112	25
13			22.80					258	306	48
A	1669.10	1659.90		106.30	2.67	9.20	96.71			
B	1659.90	1655.50		108.59	2.67	4.40	47.25			
C	1655.50	1646.30		125.16	2.67	9.20	113.87			
14	1646.30	1637.90	8.40	133.53	2.67	9.20	121.48	121	147	26
15			22.80					323	440	117
A	1637.90	1628.70		139.30	2.67	9.20	126.73			
B	1628.70	1624.30		145.67	2.67	4.40	63.38			
C	1624.30	1615.10		146.32	2.67	9.20	133.12			
16	1615.10	1606.90	8.20	154.85	3.33	9.20	175.70	176	204	28
17			23.20					487	517	30
A	1606.90	1597.70		164.53	3.33	9.20	186.69			
B	1597.70	1592.90		172.90	3.33	4.80	102.36			
C	1592.90	1583.70		174.87	3.33	9.20	198.42			
118	1583.70	1576.77	6.93	180.77	3.33	9.20	205.11	205	230	25
119	1576.77	1567.57	9.20	187.66	3.33	9.20	212.93	213	246	33
120	1567.57	1560.63	6.94	246.88	3.33	9.20	280.13	280	312	32
121	1560.63	1551.43	9.20	252.72	3.33	9.20	286.75	287	332	45
122	1551.43	1544.50	6.93	266.40	3.33	9.20	302.28	302	336	34
123	1544.50	1535.30	9.20	281.16	3.33	9.20	319.02	319	356	37

TABLE XI-2(a)

Mountain Dam Seepage Control Cutoff Wall Dimensions and Theoretical/Actual Concrete Amounts

L ,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUNT CONC. PLACED, yd3	AMOUNT OVER COMPUTED AMOUNT, yd3	PERCENT OVERAGE	PANEL NO. / BITE	
				18				1	
01	2.67	4.30	4.26					A	
09	2.67	9.20	13.73					B	
01	2.67	9.20	18.20	18	64 total 1&2	28	77	2	
				94	136	42	44	3	
07	2.67	9.20	30.09					A	
72	2.67	4.40	19.46					B	
31	2.67	9.20	44.86					C	
46	2.67	9.20	49.55	50	60	10	21	4	
				135	174	39	29	5	
43	2.67	9.20	51.34					A	
41	2.67	4.40	24.98					B	
98	2.67	9.20	58.21					C	
40	2.67	9.20	62.23	62	80	18	29	6	
				167	195	28	17	7	
00	2.67	9.20	64.59					A	
49	2.67	4.40	31.98					B	
10	2.67	9.20	70.14					C	
57	2.67	9.20	69.66	70	91	21	31	8	
				176	208	32	18	9	
10	2.67	9.20	70.14					A	
41	2.67	4.40	34.12					B	
40	2.67	9.20	72.24					C	
71	2.67	9.20	78.89	79	96	17	22	10	
				200	240	40	20	11	
.94	2.67	9.20	79.10					A	
.73	2.67	4.40	39.04					B	
39	2.67	9.20	82.23					C	
96	2.67	9.20	87.30	87	112	25	28	12	
				258	306	48	19	13	
30	2.67	9.20	96.71					A	
.59	2.67	4.40	47.25					B	
16	2.67	9.20	113.87					C	
53	2.67	9.20	121.48	121	147	26	21	14	^
				323	440	117	36	15	:
.30	2.67	9.20	126.73					A	:
.67	2.67	4.40	63.38					B	:
32	2.67	9.20	133.12					C	Type I
85	3.33	9.20	175.70	176	204	28	16	16	-------
				487	517	30	6	17	Type II
53	3.33	9.20	186.69					A	:
90	3.33	4.80	102.36					B	:
87	3.33	9.20	198.42					C	:
77	3.33	9.20	205.11	205	230	25	12	118	v
66	3.33	9.20	212.93	213	246	33	16	119	
.88	3.33	9.20	230.13	280	312	32	11	120	
72	3.33	9.20	286.75	287	332	45	16	121	
40	3.33	9.20	302.28	302	336	34	11	122	
16	3.33	9.20	319.02	319	356	37	12	123	

TABLE XI-2(a)

PANEL O. / BITE	STATION LEFT	STATION RIGHT	PANEL LENGTH,ft	PANEL DEPTH,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUN PLACED, yd3
124	1535.30	1528.37	6.93	201.73	3.33	9.20	331.02	331	361
125	1528.37	1519.17	9.20	307.41	3.33	9.20	348.81	349	390
126	1519.17	1512.23	6.94	325.95	3.33	9.20	369.84	370	412
127	1512.23	1503.03	9.20	400.26	3.33	9.20	454.16	454	524
128	1503.03	1496.10	6.93	401.08	3.33	9.20	455.09	455	503
129	1496.10	1486.90	9.20	402.55	3.33	9.20	456.76	457	516
130	1486.90	1480.07	6.83	400.52	3.33	9.20	454.46	454	502
131	1480.07	1470.87	9.20	397.90	3.33	9.20	451.48	451	488
132	1470.87	1464.05	6.82	396.58	3.33	9.20	449.99	450	500
133	1464.05	1454.85	9.20	393.70	3.33	9.20	446.72	447	496
134	1454.85	1448.02	6.83	399.08	3.33	9.20	452.82	453	502
135	1448.02	1438.82	9.20	399.50	3.33	9.20	453.30	453	499
136	1438.82	1432.00	6.82	399.93	3.33	9.20	453.79	454	504
137	1432.00	1422.80	9.20	394.52	3.33	9.20	447.65	448	486
138	1422.80	1415.97	6.83	390.51	3.33	9.20	443.10	443	494
139	1415.97	1406.77	9.20	378.77	3.33	9.20	429.78	430	468
140	1406.77	1399.95	6.82	360.89	3.33	9.20	409.49	409	460
141	1399.95	1390.75	9.20	338.58	3.33	9.20	384.18	384	423
142	1390.75	1383.92	6.83	227.36	3.33	9.20	257.98	258	308
143	1383.92	1374.72	9.20	195.21	3.33	9.20	221.50	221	264
144	1374.72	1367.90	6.82	180.44	3.33	9.20	204.74	205	232
35	1367.90	1358.70	9.20	171.59	2.67	9.20	156.11	156	202
36	1358.70	1350.30	8.40	163.06	2.67	9.20	148.35	148	176
37			21.97					329	400
A	1350.30	1341.10		155.84	2.67	9.20	141.78		
B	1341.10	1337.53		151.24	2.67	3.57	53.39		
C	1337.53	1328.33		146.65	2.67	9.20	133.42		
38	1328.33	1319.93	8.40	144.36	2.67	9.20	131.34	131	161
39			21.97					285	351
A	1319.93	1310.73		134.84	2.67	9.20	122.67		
B	1310.73	1307.16		133.53	2.67	3.57	47.14		
C	1307.16	1297.96		126.31	2.67	9.20	114.91		
40	1297.96	1289.56	8.40	126.15	2.67	9.20	114.77	115	139
41			21.97					265	336
A	1289.56	1280.36		124.67	2.67	9.20	113.42		
B	1280.36	1276.79		120.08	2.67	3.57	42.39		
C	1276.79	1267.59		120.08	2.67	9.20	109.25		
42	1267.59	1259.19	8.40	113.19	2.67	9.20	102.98	103	128
43			22.69					192	240
A	1259.19	1249.99		92.55	2.67	9.20	84.20		
B	1249.99	1245.70		88.32	2.67	4.29	37.47		
C	1245.70	1236.50		76.87	2.67	9.20	69.93		
44	1236.50	1228.10	8.40	80.38	2.67	9.20	73.13	73	84
45	1228.10	1218.90	9.20	80.05	2.67	9.20	72.83	73	102
46	1218.90	1210.50	8.40	80.87	2.67	9.20	73.57	74	92
47			23.35					187	231
A	1210.50	1201.30		80.97	2.67	9.20	73.66		
B	1201.30	1196.35		80.90	2.67	4.95	39.60		
C	1196.35	1187.15		80.97	2.67	9.20	73.66		
48	1187.15	1178.75	8.40	80.05	2.67	9.20	72.83	73	104

TABLE XI-2(b)

ANEL PTH,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUNT CONC. PLACED, yd3	AMOUNT OVER COMPUTED AMOUNT, yd3	PERCENT OVERAGE	PANEL NO. / BITE		
91.73	3.33	9.20	331.02	331	361	30	9	124		
07.41	3.33	9.20	348.81	349	390	41	12	125		
25.95	3.33	9.20	369.84	370	412	42	11	126		
00.26	3.33	9.20	454.16	454	524	70	15	127		
01.08	3.33	9.20	455.09	455	503	48	11	128		
02.55	3.33	9.20	456.76	457	516	59	13	129		
00.52	3.33	9.20	454.46	454	502	48	10	130		
97.90	3.33	9.20	451.48	451	488	37	8	131		
96.58	3.33	9.20	449.99	450	500	50	11	132		
93.70	3.33	9.20	446.72	447	496	49	11	133		
99.08	3.33	9.20	452.82	453	502	49	11	134		
99.50	3.33	9.20	453.30	453	499	46	10	135		
99.93	3.33	9.20	453.79	454	504	50	11	136		
94.52	3.33	9.20	447.65	448	486	38	9	137		
90.51	3.33	9.20	443.10	443	494	51	11	138		
78.77	3.33	9.20	429.78	430	468	38	9	139		
60.89	3.33	9.20	409.49	409	460	51	12	140		
38.58	3.33	9.20	384.18	384	423	39	10	141		
27.36	3.33	9.20	257.98	258	308	50	19	142		
95.21	3.33	9.20	221.50	221	264	43	19	143		Type II
180.44	3.33	9.20	204.74	205	232	27	13	144		-------
171.59	2.67	9.20	156.11	156	202	46	29	35		Type I
163.06	2.67	9.20	148.35	148	176	28	19	36		
				329	400	71	22	37		
155.84	2.67	9.20	141.78						A	
151.24	2.67	3.57	53.39						B	
146.65	2.67	9.20	133.42						C	
144.36	2.67	9.20	131.34	131	161	30	23	38		
				285	351	66	23	39		
134.84	2.67	9.20	122.67						A	
133.53	2.67	3.57	47.14						B	
126.31	2.67	9.20	114.91						C	
126.15	2.67	9.20	114.77	115	139	24	21	40		
				265	336	71	27	41		
124.67	2.67	9.20	113.42						A	
120.08	2.67	3.57	42.39						B	
120.08	2.67	9.20	109.25						C	
113.19	2.67	9.20	102.98	103	128	25	24	42		
				192	240	48	25	43		
92.55	2.67	9.20	84.20						A	
88.32	2.67	4.29	37.47						B	
76.87	2.67	9.20	69.93						C	
80.38	2.67	9.20	73.13	73	84	11	15	44		
80.05	2.67	9.20	72.83	73	102	29	40	45		
80.87	2.67	9.20	73.57	74	92	18	25	46		
				187	231	44	24	47		
80.97	2.67	9.20	73.66						A	
80.90	2.67	4.95	39.60						B	
80.97	2.67	9.20	73.66						C	
80.05	2.67	9.20	72.83	73	104	31	43	48		

TABLE XI-2(b)

PANEL NO. / BITE		STATION LEFT	STATION RIGHT	PANEL LENGTH,ft	PANEL DEPTH,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUNT CONC. PLACED, yd3	AMOUNT O AMOUNT,
49				23.35					185	229	4
	A	1178.75	1169.55		80.05	2.67	9.20	72.83			
	B	1169.55	1164.60		79.82	2.67	4.95	39.07			
	C	1164.60	1155.40		80.51	2.67	9.20	73.25			
50		1155.40	1147.00	8.40	80.87	2.67	9.20	73.57	74	92	1
51				19.20					152	248	9
	A	1147.00	1137.80		80.05	2.67	9.20	72.83			
	B	1137.80	1137.00		80.05	2.67	0.80	6.33			
	C	1137.00	1127.80		80.05	2.67	9.20	72.83			
52		1127.80	1119.40	8.40	81.36	2.67	9.20	74.02	74	100	2
53				21.17					170	212	4
	A	1119.40	1110.20		81.23	2.67	9.20	73.90			
	B	1110.20	1107.43		81.04	2.67	2.77	22.20			
	C	1107.43	1098.23		81.36	2.67	9.20	74.02			
54		1098.23	1089.83	8.40	73.82	2.67	9.20	67.16	67	76	
55				21.17					148	160	1
	A	1089.83	1080.63		70.87	2.67	9.20	64.48			
	B	1080.63	1077.86		70.61	2.67	2.77	19.34			
	C	1077.86	1068.66		70.87	2.67	9.20	64.48			
56		1068.66	1060.26	8.40	55.38	2.67	9.20	50.38	50	64	1
57				20.77					122	136	1
	A	1060.26	1051.06		59.34	2.67	9.20	53.99			
	B	1051.06	1048.69		59.05	2.67	2.37	13.84			
	C	1048.69	1039.49		59.71	2.67	9.20	54.32			

807.51 -Total Length of Cutoff Wall at Widest Point

	Computed Total,yd3	Actually Placed,yd3	Overa
Cutoff Wall	15,751	18,277	2,51
Type I / Left Sta. 16+15 to 18+47	1,859	2,349	49
Type I / Right Sta. 10+39.5 to 13+69	3,246	4,063	8
Type II / Canyon Sta. 13+69 to 16+15	10,646	11,865	1,2

Spillway section. The bottom of the trench was approximately 80 feet from top of the ramp built across the spillway surface. The panel depth figures reflect the depth of concrete placed.

All panel depths are the sounded depths which meet or exceed the approved depth

TABLE XI-2(c)

PANEL DEPTH,ft*	PANEL WIDTH,ft	PANEL BITE LENGTH,ft	COMPUTED AMT. CONC. PER BITE, yd3	COMPUTED AMOUNT CONC./PANEL, yd3	ACTUAL AMOUNT CONC. PLACED, yd3	AMOUNT OVER COMPUTED AMOUNT, yd3	PERCENT OVERAGE	PANEL NO / BITE
				185	229	44	24	49
80.05	2.67	9.20	72.83					A
79.82	2.67	4.95	39.07					B
80.51	2.67	9.20	73.25					C
80.87	2.67	9.20	73.57	74	92	18	25	50
				152	248	96	63	51
80.05	2.67	9.20	72.83					A
80.05	2.67	0.80	6.33					B
80.05	2.67	9.20	72.83					C
81.36	2.67	9.20	74.02	74	100	26	35	52
				170	212	42	25	53
81.23	2.67	9.20	73.90					A
81.04	2.67	2.77	22.20					B +
81.36	2.67	9.20	74.02					C +
73.82	2.67	9.20	67.16	67	76	9	13	54 +
				148	160	12	8	55 +
70.87	2.67	9.20	64.48					A +
70.61	2.67	2.77	19.34					B +
70.87	2.67	9.20	64.48					C +
55.38	2.67	9.20	50.38	50	64	14	27	56 +
				122	136	14	11	57 +
59.34	2.67	9.20	53.99					A +
59.05	2.67	2.37	13.84					B +
59.71	2.67	9.20	54.32					C +

		Computed Total,yd3	Actually Placed,yd3	Overage,yd3	Average Percent Over
-Total Length of Cutoff Wall at Widest Point	Cutoff Wall	15,751	18,277	2,526	16
	Type I / Left Sta. 16+15 to 18+47.	1,859	2,349	490	26
	Type I / Right Sta. 10+39.5 to 13+69	3,246	4,063	817	25
ench was amp built depth figures					
	Type II / Canyon Sta. 13+69 to 16+.5	10,646	11,365	1,019	[illegible]
which					

TABLE XI-2(c)

MUD MOUNTAIN DAM CUTOFF WALL TREMIE CONCRETE DATA

PANEL	CONCRETE cu. yds.	PANEL AVERAGES W/C	SLUMP in.	% AIR	UNIT WT lbs/ft3	COMP. STRENGTH, PSI 7-DAY	28-DAY
1 & 2	64	0.40	8.00	3.7	147.7	2980	4950
3	136	0.43	7.88	4.7	146.8	3210	5400
4	60	0.43	7.58	3.8	147.6	2990	4810
5	174	0.41	7.53	3.7	148.6	3500	5510
6	80	0.44	8.33	4.1	146.0	3060	4670
7	195	0.45	8.80	3.1	149.1	2430	4550
8	91	0.44	8.38	4.6	147.0	3070	5210
9	208	0.46	8.69	3.1	147.8	2680	4830
10	96	0.44	8.50	4.2	147.7	2190	4420
11	240	0.45	8.87	3.4	148.7	2500	4690
12	112	0.43	8.33	4.9	146.9	3680	5420
13	306	0.47	7.57	3.7	148.7	3000	4740
14	147	0.43	8.10	4.6	146.9	3050	5210
15	440	0.48	9.44	2.0	149.4	2200	4390
16	204	0.43	8.39	5.0	146.6	3430	5610
17	517	0.44	8.76	3.5	148.5	2540	4680
118	230	0.44	9.00	3.3	146.8	2890	4720
119	246	0.44	8.38	3.1	148.7	3310	5500
120	312	0.44	8.61	4.0	146.4	2860	5030
121	332	0.45	7.95	3.6	148.4	2690	4980
122	336	0.45	8.70	4.0	146.1	3230	5470
123	356	0.45	8.50	3.2	149.2	3420	5150
124	361	0.44	8.66	3.3	146.9	3010	4950
125	390	0.46	7.88	5.1	147.4	2510	4640
126	412	0.45	8.46	3.7	146.9	3030	5240
127	524	0.43	8.33	3.5	147.9	3290	5600
128	503	0.44	8.15	4.0	148.2	3540	5880
129	516	0.45	8.51	3.4	147.2	3050	5140
130	502	0.44	8.46	3.8	147.6	3780	5820
131	488	0.46	7.85	3.5	147.3	2630	5230
132	500	0.44	8.00	3.4	146.8	3300	5620
133	496	0.46	8.01	3.4	147.3	2970	5230
134	502	0.44	8.27	3.0	148.5	3850	5810
135	499	0.43	8.55	3.3	147.9	3470	5600
136	504	0.45	7.91	3.7	146.8	3400	5710
137	486	0.45	8.57	3.4	147.3	2870	5160
138	494	0.44	7.98	3.5	148.5	3490	5780
139	468	0.45	8.48	3.7	146.9	3210	5140
140	460	0.42	8.67	3.7	147.8	3600	5640
141	423	0.45	8.45	3.6	146.9	3080	5180
142	308	0.45	8.53	4.0	146.6	3190	5690
143	264	0.42	8.13	4.4	146.2	3270	5390
144	232	0.43	8.08	4.0	147.1	2970	4910
35	202	0.43	8.22	4.4	147.0	3680	5550
36	176	0.43	8.47	4.2	147.0	3370	5430
37	400	0.42	8.56	4.8	146.9	3410	5020
38	161	0.43	7.96	4.6	146.3	2760	4980
39	351	0.44	8.64	5.5	146.0	3190	5050
40	139	0.41	8.30	4.4	146.1	2790	5080

TABLE XI-3

PANEL	CONCRETE cu. yds.	W/C	SLUMP in.	% AIR	UNIT WT lbs/ft3	COMP. STRENGTH, PSI 7-DAY	28-DAY
			PANEL AVERAGES				
41	336	0.43	8.63	4.7	146.5	3420	5200
42	128	0.42	8.00	3.7	147.9	3410	5750
43	240	0.45	7.73	4.9	146.9	3330	5120
44	84	0.42	8.25	4.1	146.9	3040	5100
45	102	0.42	8.59	4.1	147.6	3110	4980
46	92	0.42	8.13	3.6	148.0	3450	5590
47	231	0.44	7.36	5.2	146.9	3300	5470
48	104	0.41	8.67	5.1	145.2	3010	5130
49	229	0.43	8.84	3.5	148.1	2670	4720
50	92	0.42	8.10	4.1	146.6	3320	5190
51	248	0.42	8.44	4.4	147.4	3220	5190
52	100	0.43	8.63	4.2	146.3	2920	4820
53	212	0.41	8.25	4.5	146.2	2910	5340
54	76	0.44	8.67	4.0	146.3	2600	4720
55	160	0.42	8.44	4.5	147.0	3230	5180
56	64	0.43	8.25	4.3	146.4	2670	4650
57	136	0.42	8.45	4.1	146.7	2740	4990
	18277 TOTAL						

	W/C	SLUMP	% AIR	UNIT WT	7-DAY	28-DAY
Overall Average	0.44	8.34	3.8	147.4	3130	5240
Range - Low	0.40	3.25	0.5	141.2	1620	3610
High	0.51	10.00	7.2	151.3	4730	7290
Number of Tests	440	671	421	441	440	877
Standard Deviation	0.02	0.77	0.81	1.33	500	551
Coefficient of Variation, %	4.14	9.21	21.16	0.90	16.00	10.52

TABLE XI-4

Seepage Control Cutoff Wall Cast-In-Place Extension Stationing

Panel Number	Station Left	Station Right	Panel Length, ft.
1	1847.00	1827.00	20.00
2	1827.00	1807.00	20.00
3	1807.00	1787.00	20.00
4	1787.00	1767.00	20.00
5	1767.00	1747.00	20.00
6	1747.00	1727.00	20.00
7	1727.00	1707.00	20.00
8	1707.00	1687.00	20.00
9	1687.00	1667.00	20.00
10	1667.00	1647.00	20.00
11	1647.00	1627.00	20.00
12	1627.00	1607.00	20.00
13	1607.00	1587.00	20.00
14	1587.00	1567.00	20.00
15	1567.00	1547.00	20.00
16	1547.00	1527.00	20.00
17	1527.00	1507.00	20.00
18	1507.00	1487.00	20.00
19	1487.00	1467.00	20.00
20	1467.00	1447.00	20.00
21	1447.00	1437.00	10.00
22	1437.00	1417.00	20.00
23	1417.00	1407.00	10.00
24	1407.00	1387.00	20.00
25	1387.00	1367.00	20.00
26	1367.00	1347.00	20.00
27	1347.00	1327.00	20.00
28	1327.00	1307.00	20.00
29	1307.00	1287.00	20.00
30	1287.00	1267.00	20.00
31	1267.00	1250.00	17.00
32	1250.00	1230.00	20.00
33	1230.00	1210.00	20.00
34	1210.00	1190.00	20.00
35	1190.00	1170.00	20.00
36	1170.00	1150.00	20.00
37	1150.00	1141.78	8.22
38	1141.78	1132.78	9.00
			714.22 -Total Length

TABLE XI-5

Seepage Control Cutoff Wall Cast-In-Place Extension
Concrete Data

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	PANEL	DATE	CY PLACED
0.42	3.75	4.4	2,500	5,440	5,390	148.9	54	58	8,10,12	22-May-90	33.0
			2,430						8,10,12	22-May-90	
			1,860						8,10,12	22-May-90	
0.42	2.75	4.1	2,790	5,430	5,410	148.4	54	58	8,10,12	22-May-90	
			2,790						8,10,12	22-May-90	
			2,830						8,10,12	22-May-90	
0.48	4.00	5.4	2,870	5,140	5,120	152.4	60	58	7,9,11,13	24-May-90	44.0
0.51	2.50	4.2	3,720	5,820	5,790	148.6	72	62	31,33,35	30-May-90	34.0
0.46	5.00	5.2	2,660	4,510	4,580	146.4	54	58	29,32,34,36	01-Jun-90	44.0
0.45	2.50	5.0	2,960	5,080	5,340	147.7	62	64	24,26,28,30	05-Jun-90	44.0
0.46	2.75	4.2	3,750	5,610	5,550	149.2	64	66	20,22,25,27	07-Jun-90	44.5
0.47	2.50	5.0	3,400	5,080	4,920	147.8	62	64	16,18,21,23	13-Jun-90	36.0
0.49	3.00	5.4	3,030	4,440	4,370	146.9	75	70	17,19	14-Jun-90	22.0
0.49	2.75	5.4	3,020			147.0	75	70	17,19	14-Jun-90	
			2,990						17,19	14-Jun-90	
			3,140						17,19	14-Jun-90	
0.48	2.50	5.0	3,440	5,180	5,220	147.2	73	70	2,4,6	19-Jun-90	34.0
0.48	2.75	4.9	3,300	5,010	5,020	147.2	75	71	1,3,5,37	21-Jun-90	37.0
0.44	2.00	4.7	3,840	5,160	5,170	147.5	86	84	14,15	03-Aug-90 *	22.0
0.51	2.25	4.7	3,090	4,390	4,470	146.5	86	86	14,15	03-Aug-90	
0.51									14,15	03-Aug-90	
0.50									14,15	03-Aug-90	
0.49									14,15	03-Aug-90	
0.43	2.75	5.7	2,650	4,040	4,090	144.0	78	80	38	07-Aug-90	8.0
											402.5

* 4-DAY	5-DAY	6-DAY
3,080	3,210	3,670

TABLE XI-6

Seepage Control Cutoff Wall Cast-In-Place Extension
Concrete Data

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	
0.47	2.92	4.9	3003	5028	147.7	69	68	Overall Average
0.03	0.77	0.5	475	490	2	11	9	Standard Deviation
6.4	26.5	10.2	15.8	9.7	1.2	15.9	13.7	Coefficient of Variation, %
0.51	5.00	5.7	3940	5820	152.4	86	86	Maximum
0.42	2.00	4.1	1860	4040	144.0	54	58	Minimum
18	15	15	21	28	15	15	15	Number of Tests

AVG 7-DAY	AVG 28-DAY	PANEL
2533	5418	8,10,12
2870	5130	7,9,11,13
3720	5805	31,33,35
2660	4545	29,32,34,36
2960	5210	24,26,28,30
3750	5580	20,22,25,27
3400	5000	16,18,21,23
3045	4405	17,19
3440	5200	2,4,6
3300	5015	1,3,5,37
3465	4798	14,15
2650	4065	38

Design Strength Specified: 3,000 psi @ 28-Days

TABLE XI-7

Spillway Sections Replacement
Concrete Data

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	PANEL	DATE	CY PLACED
0.43	2.75	5.2	3,280	4,530	4,580	147.0	66	76	1,4	17-Sep-90	70
0.43	2.75	5.4	3,470	4,600	4,650	147.1	70	79	1,4	17-Sep-90	
0.43	2.50	5.2	3,950	4,970	5,000	148.5	64	72	2,3	18-Sep-90	130
0.44	2.25	5.0	4,210	5,270	5,240	148.0	68	76	2,3	18-Sep-90	
0.45	2.75	5.0	4,120	5,090	5,041	148.4	72	79	2,3	18-Sep-90	
0.42	2.25	5.2	3,670	4,970	4,930	146.5	51	64	Slot	02-Nov-90	27
											227

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	
0.43	2.54	5.2	3,783	4,906	147.6	65	74	Overall Average
0.01	0.25	0.2	371	256	1	7	6	Standard Deviation
2.4	9.7	2.9	9.8	5.2	0.6	11.5	7.6	Coefficient of Variation, %
0.45	2.75	5.4	4,210	5,270	148.5	72	79	Maximum
0.42	2.25	5.0	3,280	4,530	146.5	51	64	Minimum
6	6	6	6	12	6	6	6	Number of Tests

Design Strength Specified: 4,000 psi @ 28-Days

TABLE XI-8

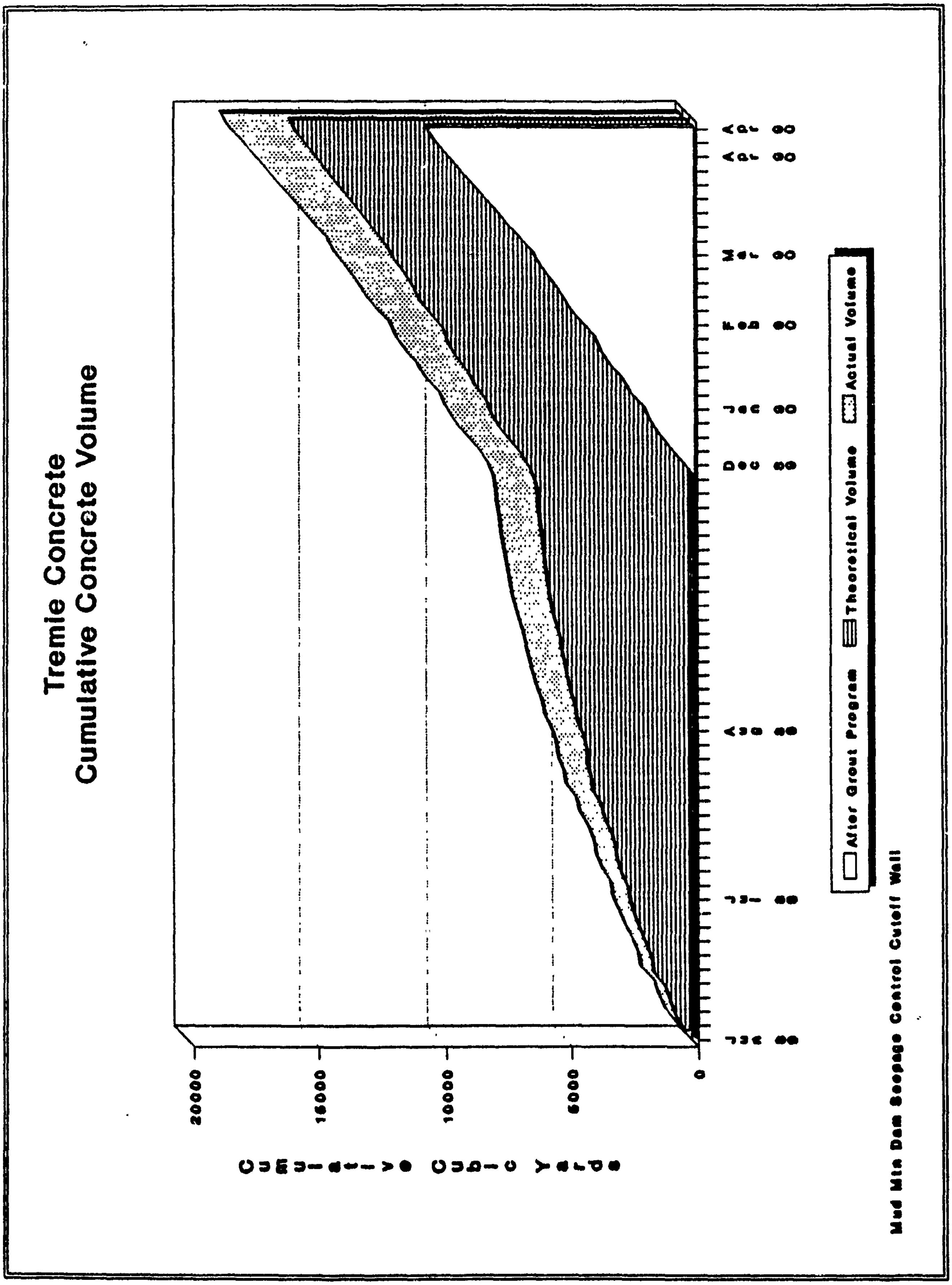
Lower Cascade Creek Diversion Basin Concrete Data

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	LOCATION	DATE	CY PLACED
0.41	6.50	8.0	2,300	3,200	3,220	138.5	64	83	Head Wall	20-Sep-90	3.5
0.36	6.25	6.2	2,540	3,270	3,240	146.8	74	80	Head Wall	20-Sep-90	
0.43	2.50	5.2	3,310	4,570	4,600	147.0	61	73	Pad	25-Sep-90	2.0
0.45	2.25	4.9	3,440	5,590	5,610	149.6	64	74	Basin	27-Sep-90	31.5
			2,970	4,940					Basin	27-Sep-90	
0.41	2.00	4.8	4,300	5,400	5,400	148.3	56	68	Wings	03-Oct-90	2.5
											39.5

W/C	SLUMP in.	% AIR	7-DAY lbs/in2	28-DAY lbs/in2	UNIT WT lbs/ft3	TEMPERATURE, DEG. F AIR	CONCRETE	
0.41	3.90	5.8	3,143	4,464	146.0	64	76	Overall Average
0.03	2.27	1.3	715	1,038	4	7	6	Standard Deviation
8.1	58.2	23.0	22.8	23.3	3.0	10.3	7.9	Coefficient of Variation, %
0.45	6.50	8.0	4,300	5,610	149.6	74	83	Maximum
0.36	2.00	4.8	2,300	3,200	138.5	56	68	Minimum
5	5	5	6	11	5	5	5	Number of Tests

Design Strength Specified: 3,000 psi @ 28-Days

TABLE XI-9



Tremie Concrete
Cumulative Concrete Volume
Cumulative Cubic Yards
20000
15000
10000
5000
0
After Grout Program
Theoretical Volume
Actual Volume
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI-1

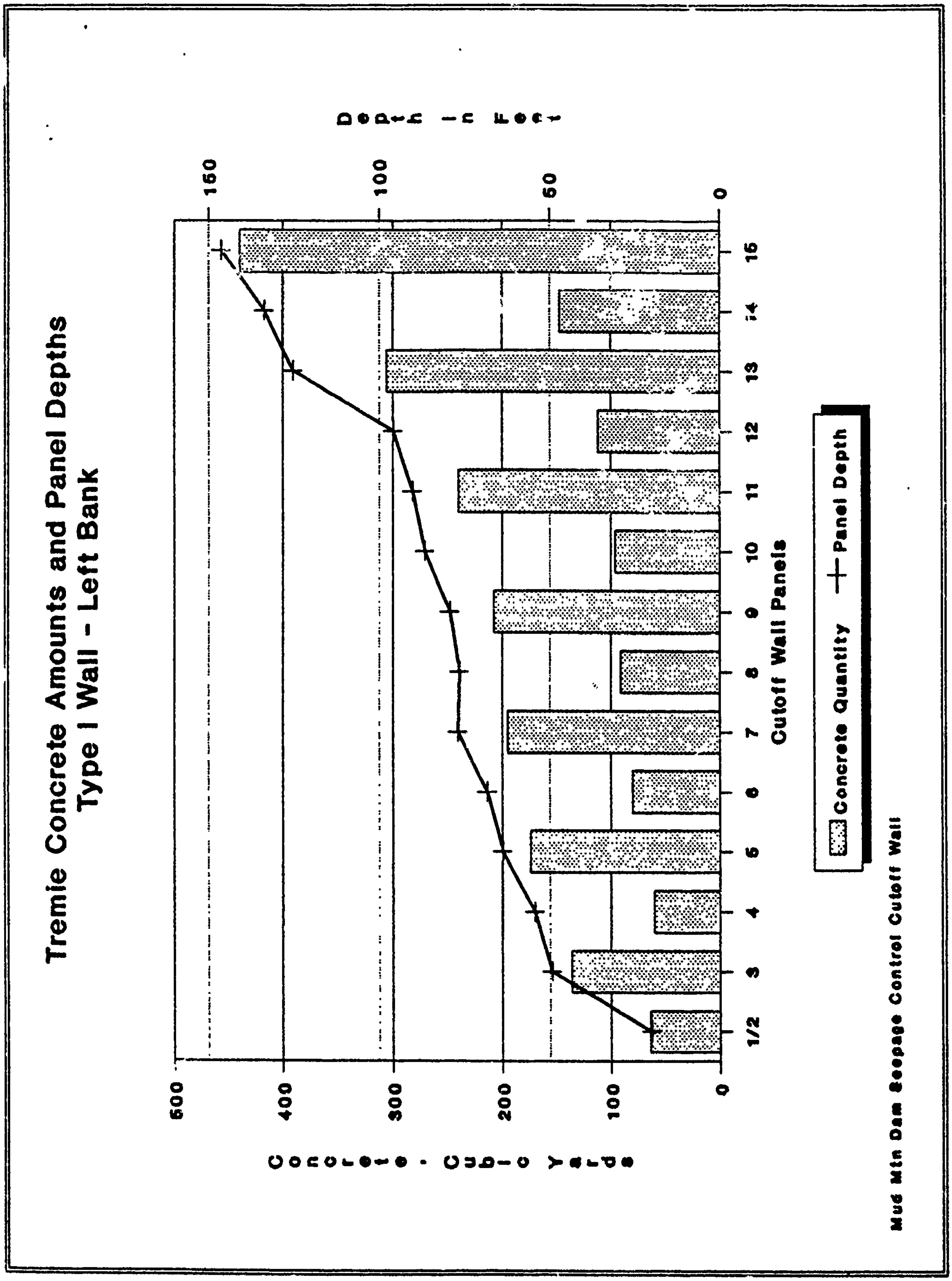

Tremie Concrete Amounts and Panel Depths
Type I Wall - Left Bank
Concrete - Cubic Yards
Depth in Feet
Cutoff Wall Panels
Concrete Quantity
Panel Depth
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI-2

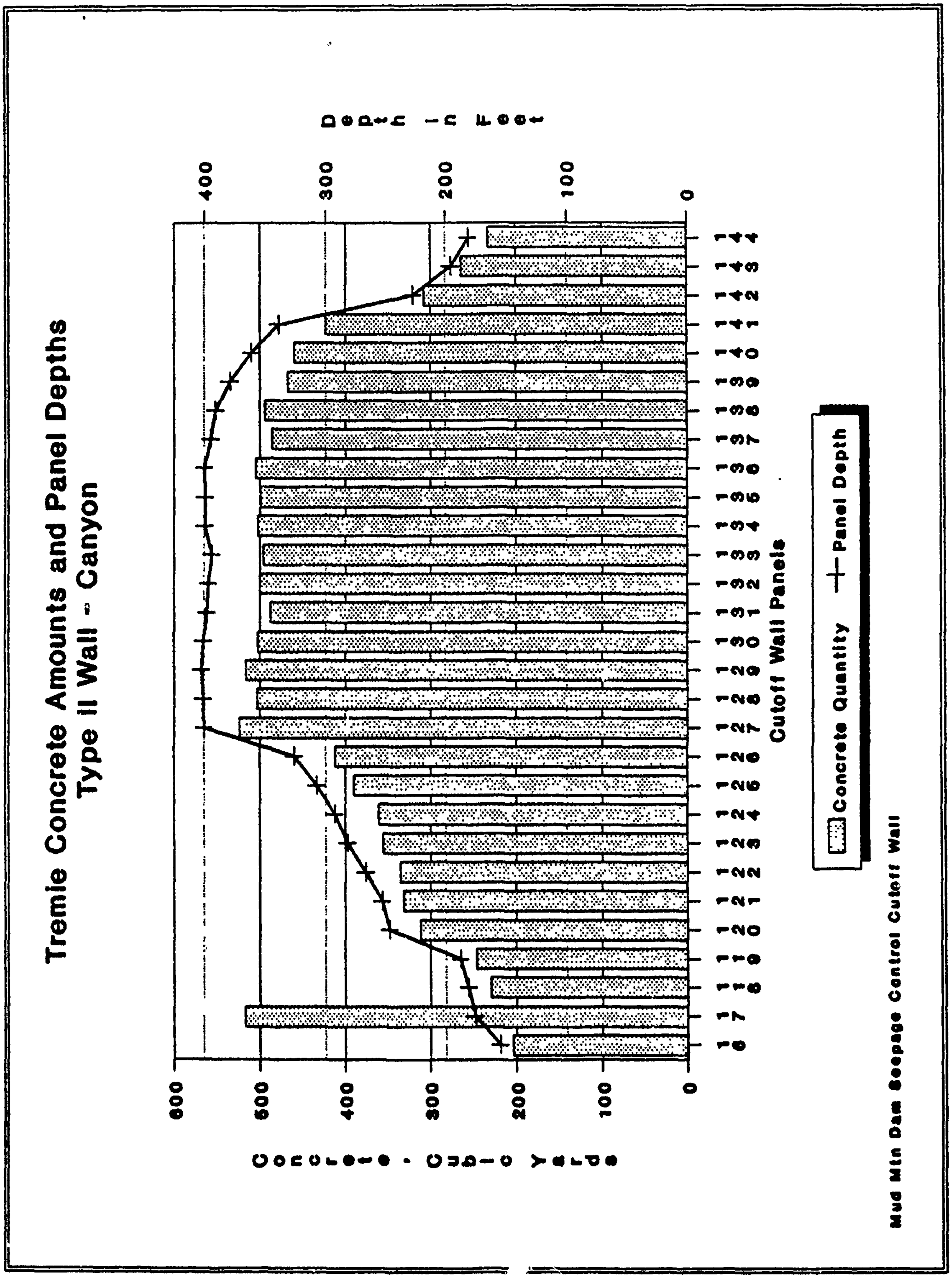

Tremie Concrete Amounts and Panel Depths
Type II Wall - Canyon
Depth in Feet
400
300
200
100
0
Concrete - Cubic Yards
Cutoff Wall Panels
116 117 118 119 120 121 122 123 124 125 126 127 128 129 130 131 132 133 134 135 136 137 138 139 140 141 142 143 144
Concrete Quantity
Panel Depth
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI-3

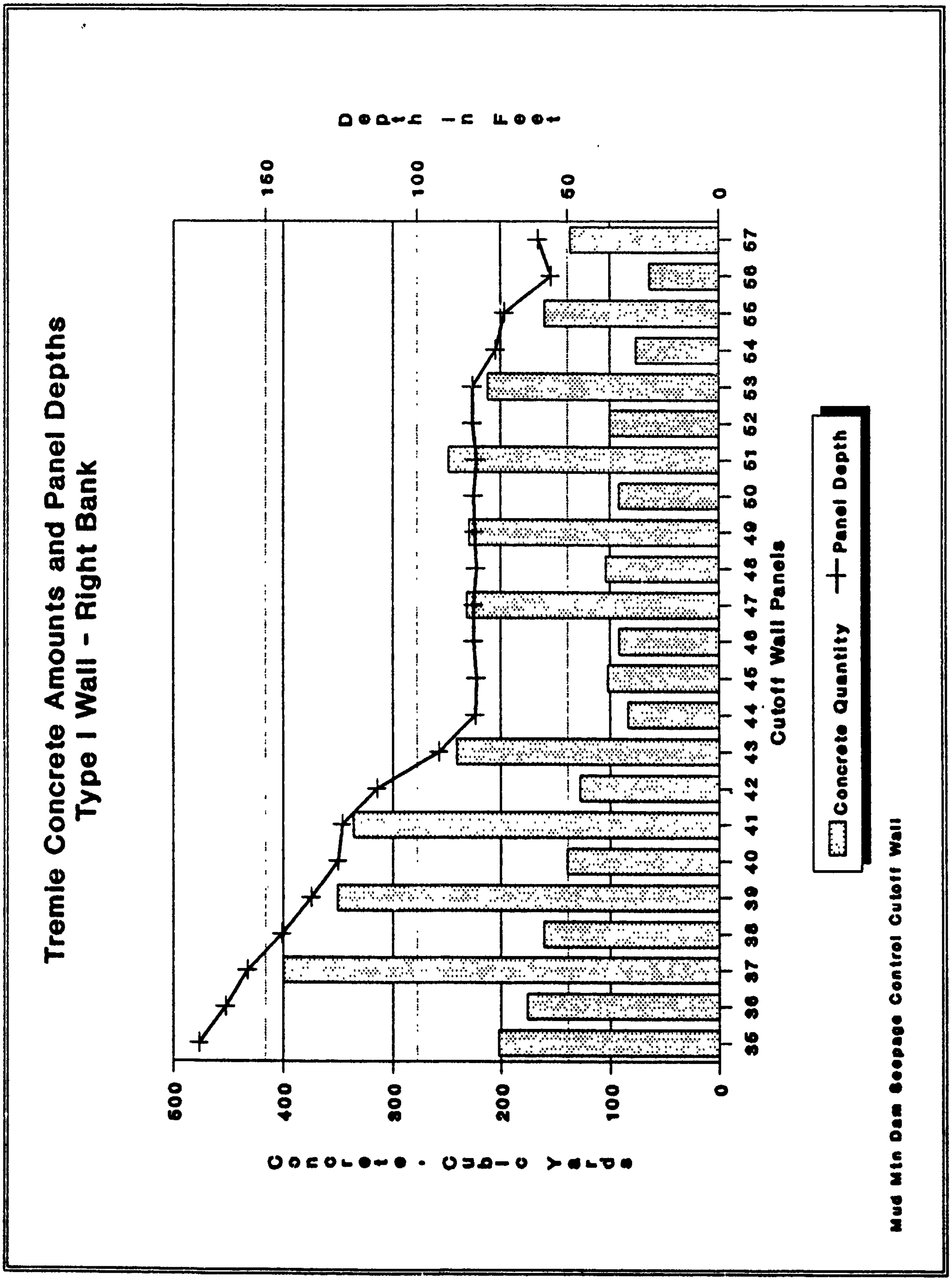

Tremie Concrete Amounts and Panel Depths
Type I Wall - Right Bank
Concrete - Cubic Yards
Depth in Feet
Cutoff Wall Panels
Concrete Quantity
Panel Depth
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI-4

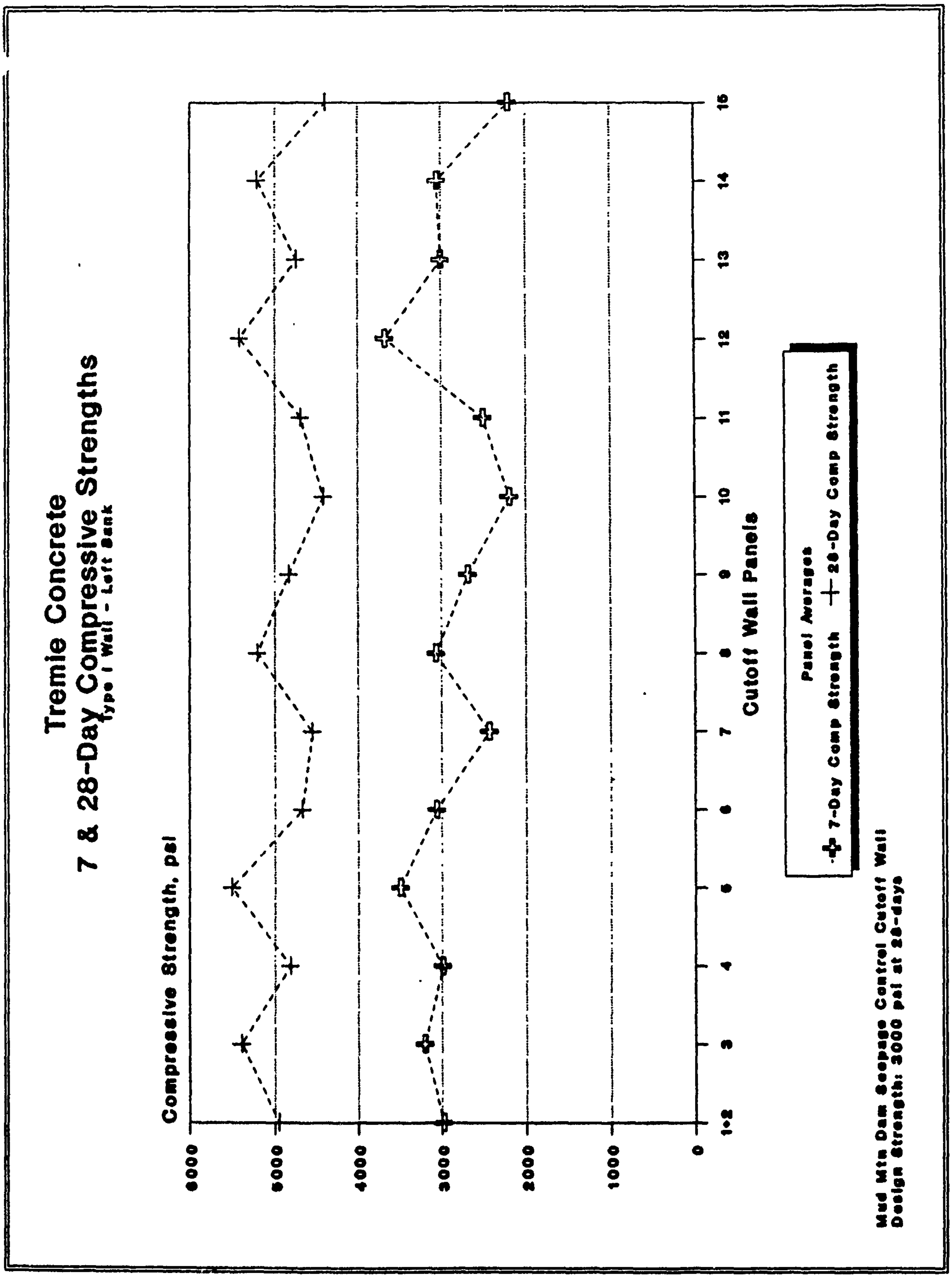

Tremie Concrete
7 & 28-Day Compressive Strengths
Type I Wall - Left Bank
Compressive Strength, psi
6000
5000
4000
3000
2000
1000
0
1-2
3
4
5
6
7
8
9
10
11
12
13
14
15
Cutoff Wall Panels
Panel Averages
7-Day Comp Strength
28-Day Comp Strength
Mud Mtn Dam Seepage Control Cutoff Wall
Design Strength: 3000 psi at 28-days

FIGURE XI-5

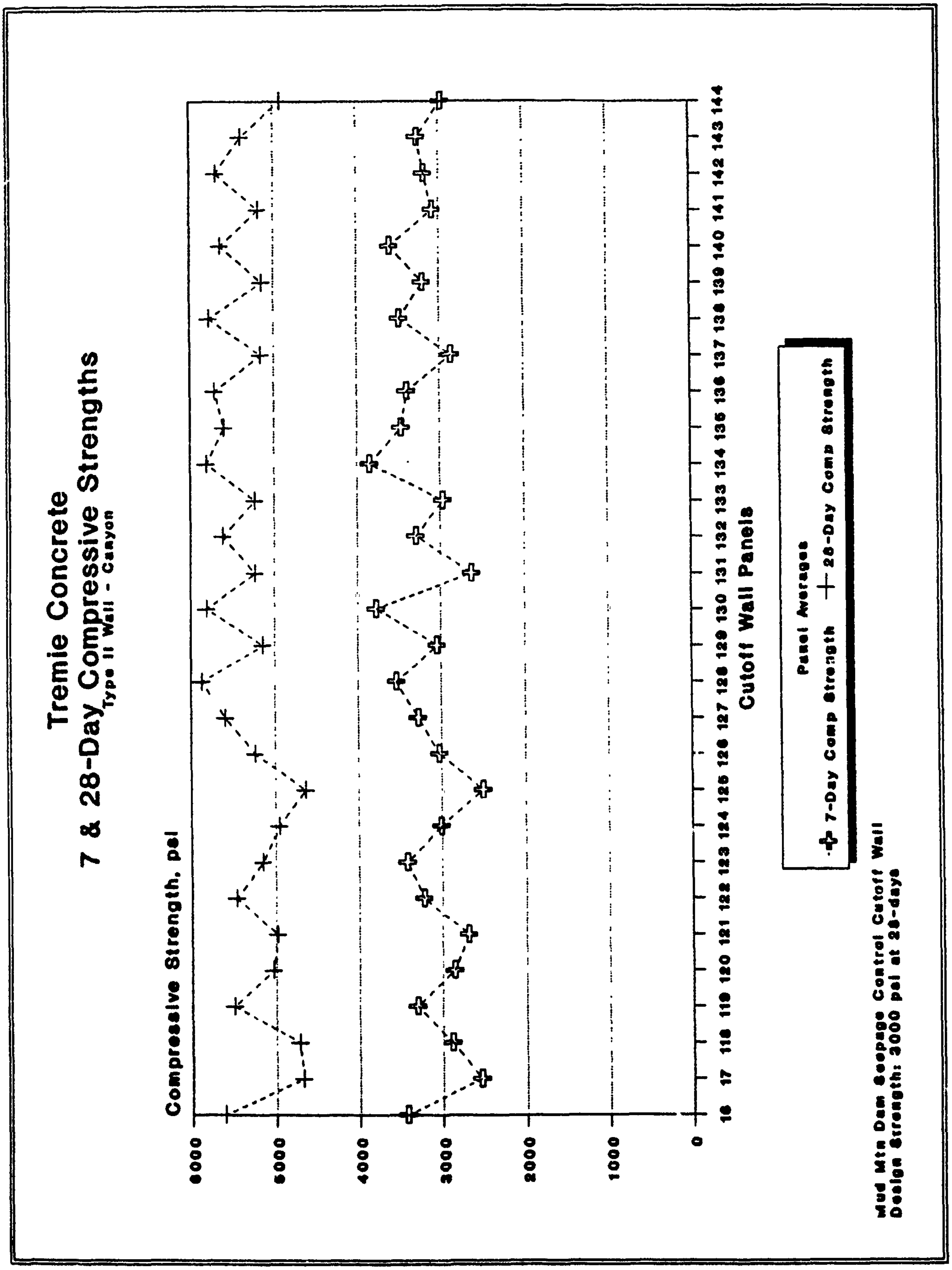
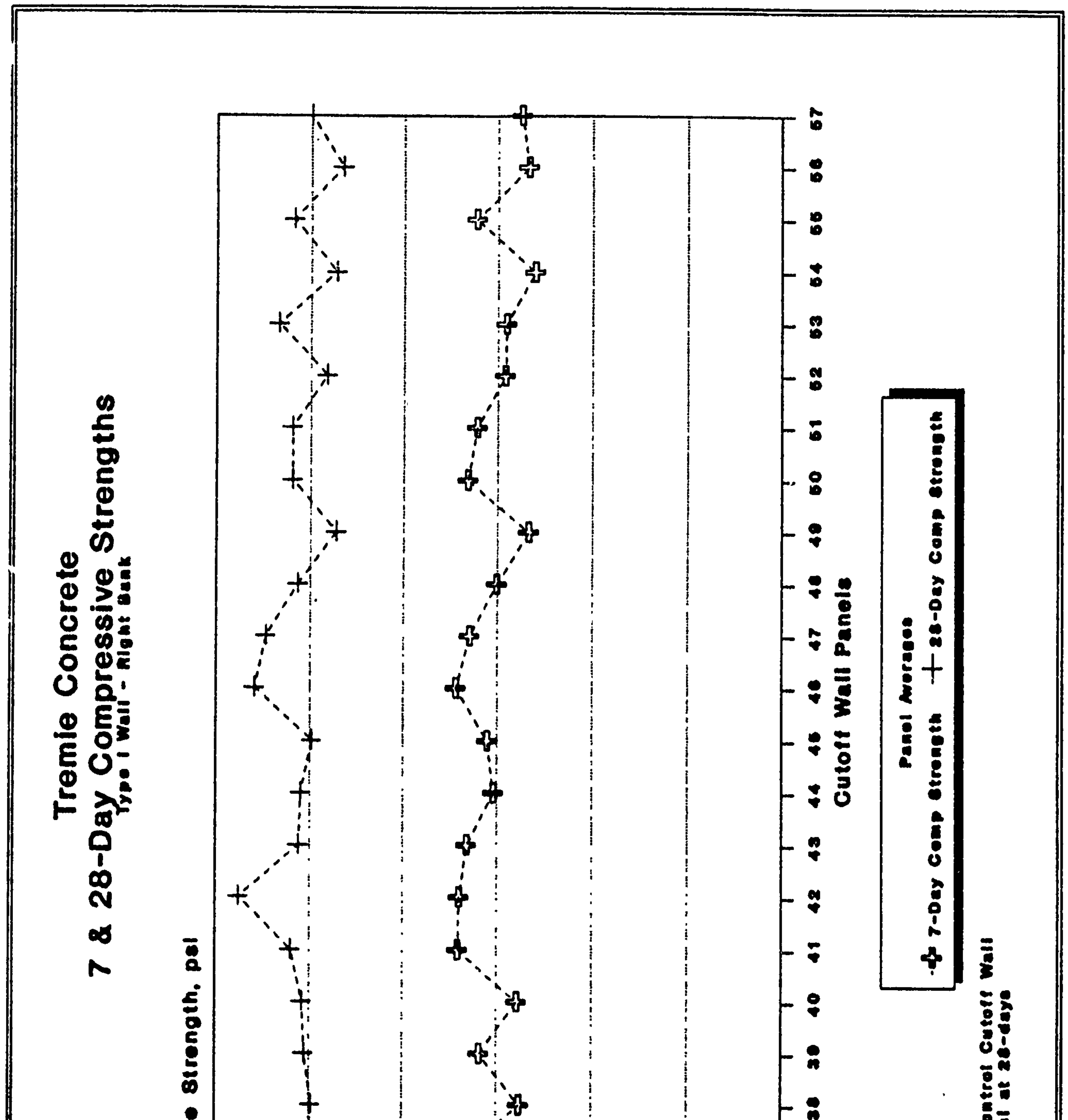
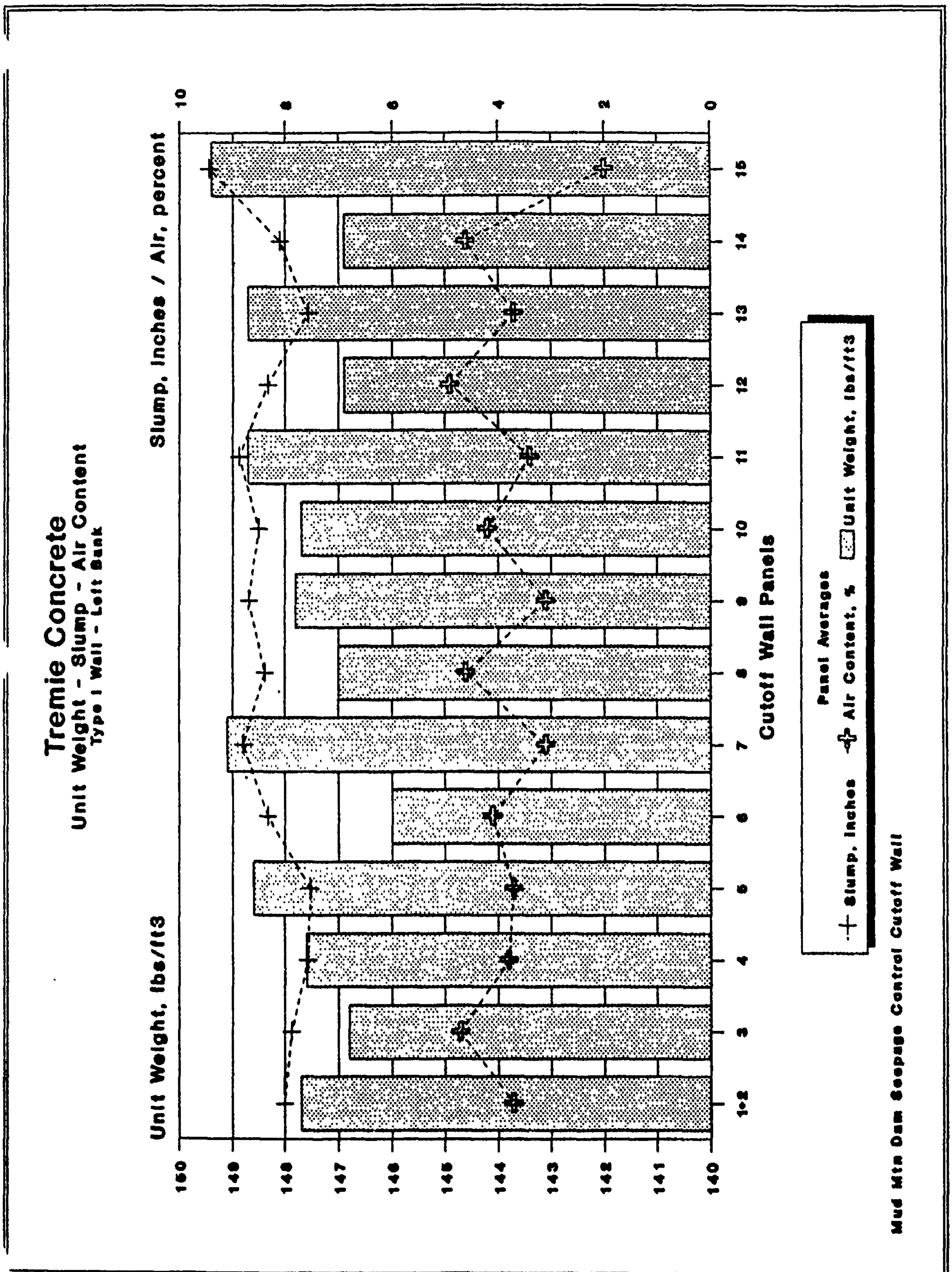

Tremie Concrete
7 & 28-Day Compressive Strengths
Type II Wall - Canyon
Compressive Strength, psi
6000
5000
4000
3000
2000
1000
0
16 17 118 119 120 121 122 123 124 125 126 127 128 129 130 131 132 133 134 135 136 137 138 139 140 141 142 143 144
Cutoff Wall Panels
Panel Averages
7-Day Comp Strength
28-Day Comp Strength
Mud Mtn Dam Seepage Control Cutoff Wall
Design Strength: 3000 psi at 28-days

FIGURE XI-6



Tremie Concrete
7 & 28-Day Compressive Strengths
Type I Wall - Right Bank
Compressive Strength, psi
6000
5000
4000
3000
2000
1000
0
35
36
37
38
39
40
41
42
43
44
45
46
47
48
49
50
51
52
53
54
55
56
57
Cutoff Wall Panels
Panel Averages
7-Day Comp Strength
28-Day Comp Strength
Mud Mtn Dam Seepage Control Cutoff Wall
Design Strength: 3000 psi at 28-days

FIGURE XI-7



Tremie Concrete
Unit Weight - Slump - Air Content
Type I Wall - Left Bank
Unit Weight, lbs/ft3
Slump, inches / Air, percent
150
149
148
147
146
145
144
143
142
141
140
10
8
6
4
2
0
1+2
3
4
5
6
7
8
9
10
11
12
13
14
15
Cutoff Wall Panels
Panel Averages
Slump, inches
Air Content, %
Unit Weight, lbs/ft3
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI- 8

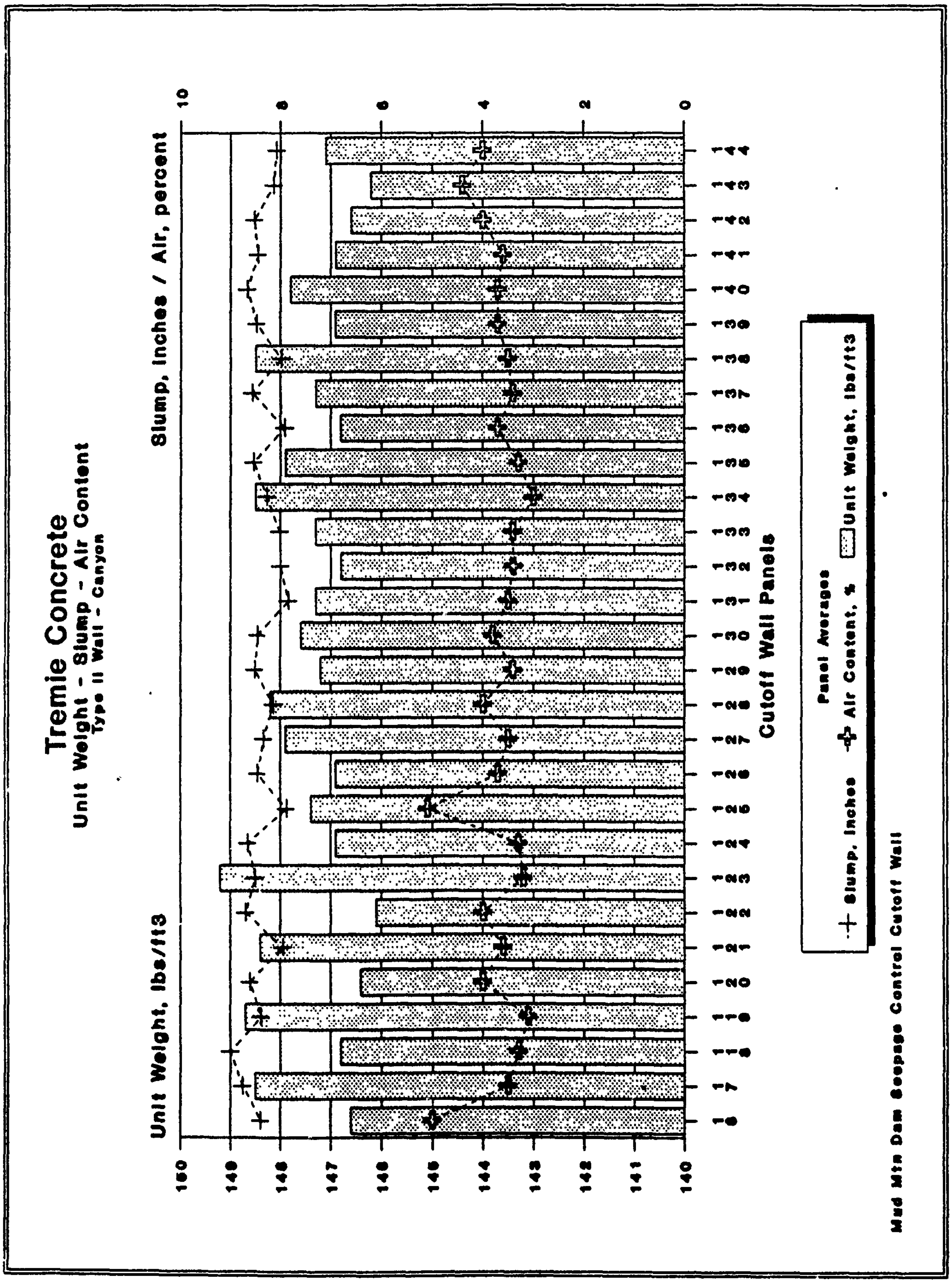

Tremie Concrete
Unit Weight - Slump - Air Content
Type II Wall - Canyon
Unit Weight, lbs/ft3
Slump, inches / Air, percent
Cutoff Wall Panels
Panel Averages
Slump, inches
Air Content, %
Unit Weight, lbs/ft3
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI- 9

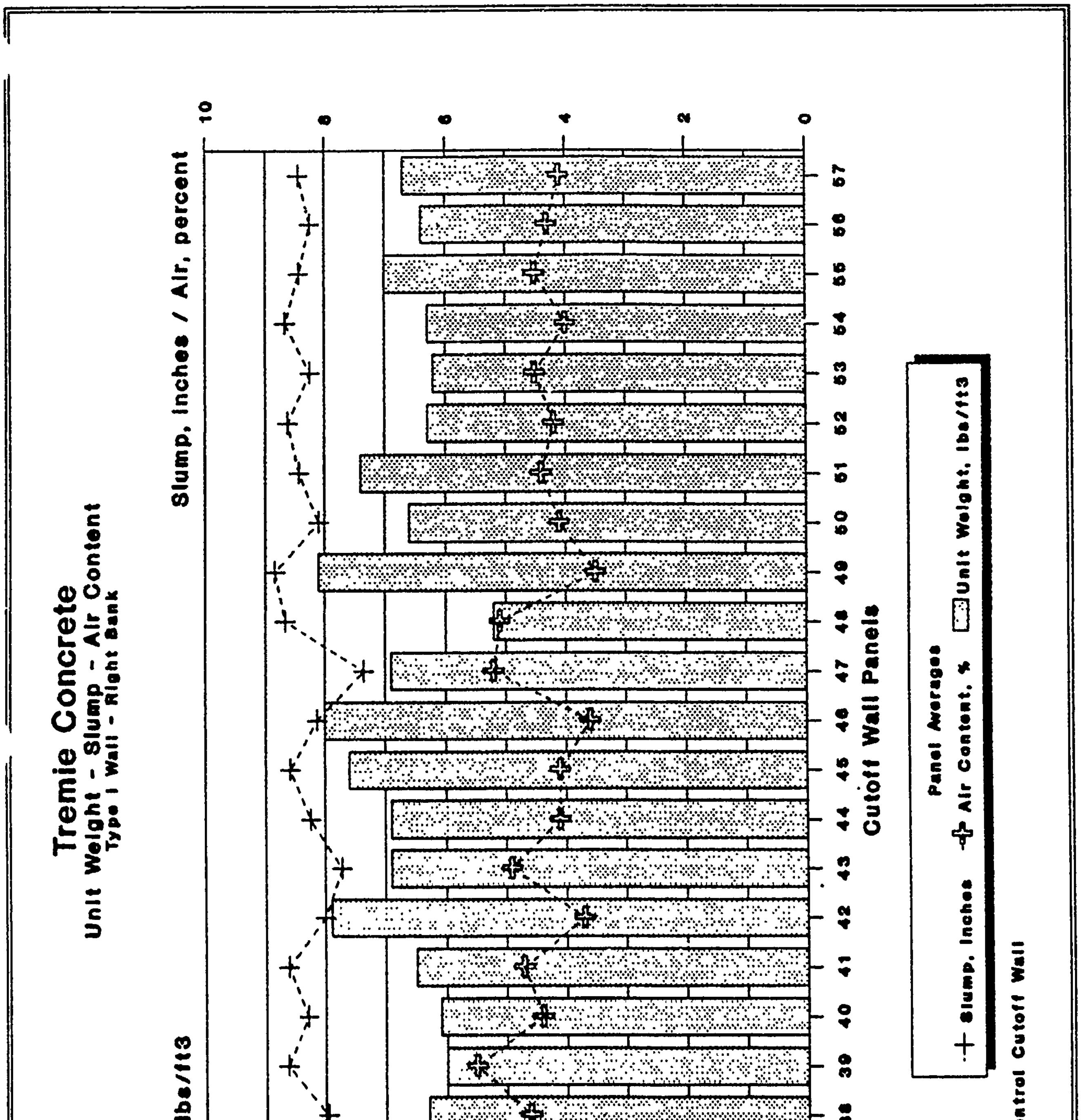

Tremie Concrete
Unit Weight - Slump - Air Content
Type I Wall - Right Bank
Unit Weight, lbs/ft3
Slump, inches / Air, percent
150
149
148
147
146
145
144
143
142
141
140
10
8
6
4
2
0
36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57
Cutoff Wall Panels
Panel Averages
Slump, inches
Air Content, %
Unit Weight, lbs/ft3
Mud Mtn Dam Seepage Control Cutoff Wall

FIGURE XI- 10

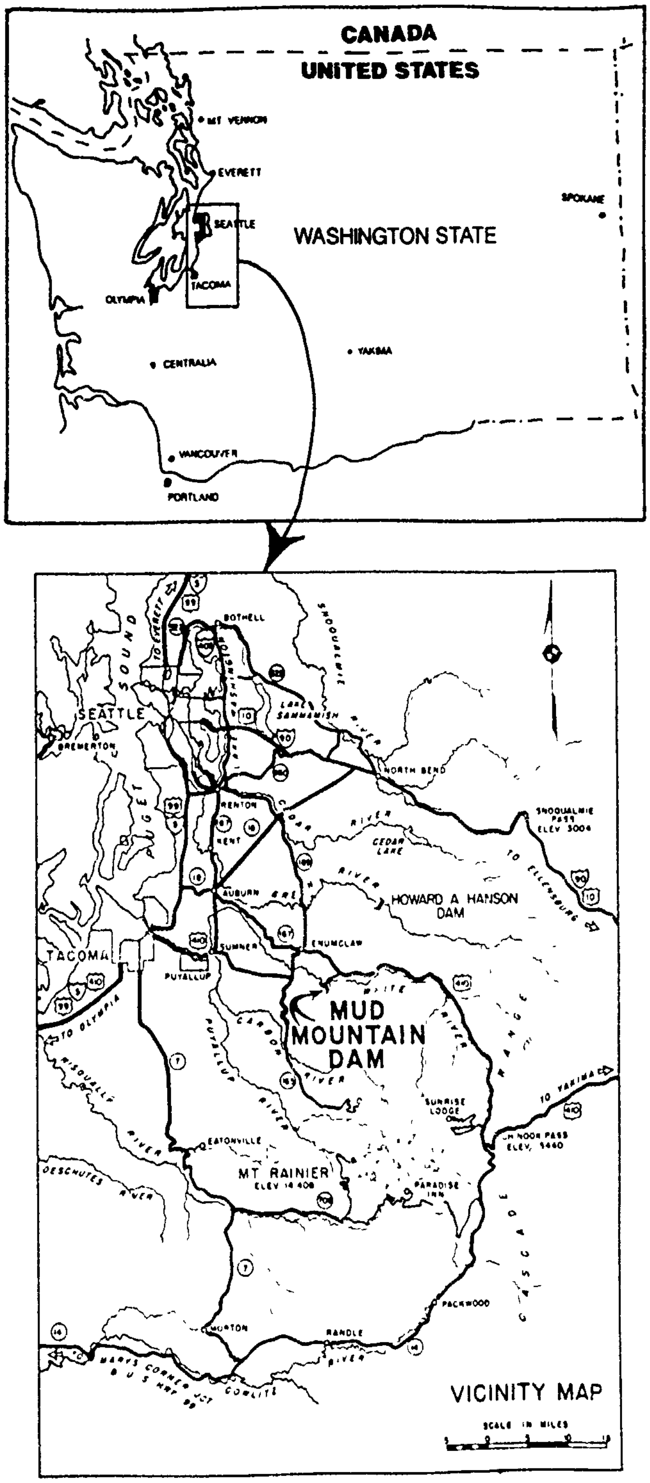
MUD MOUNTAIN DAM CUTOFF WALL CONSTRUCTUION FOUNDATION REPORT

3 | 2 | 1

NDATION REPORT

TION

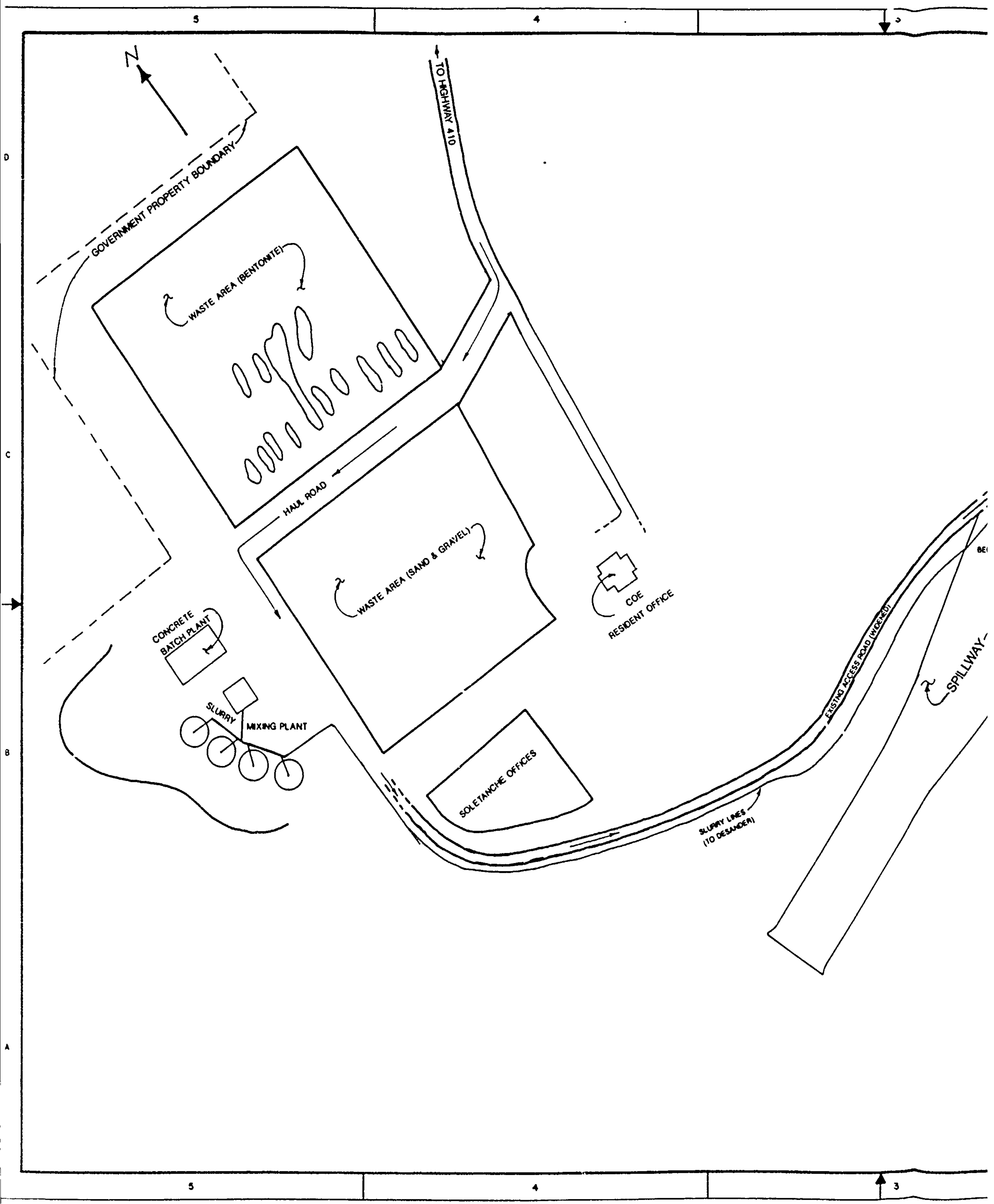
SION



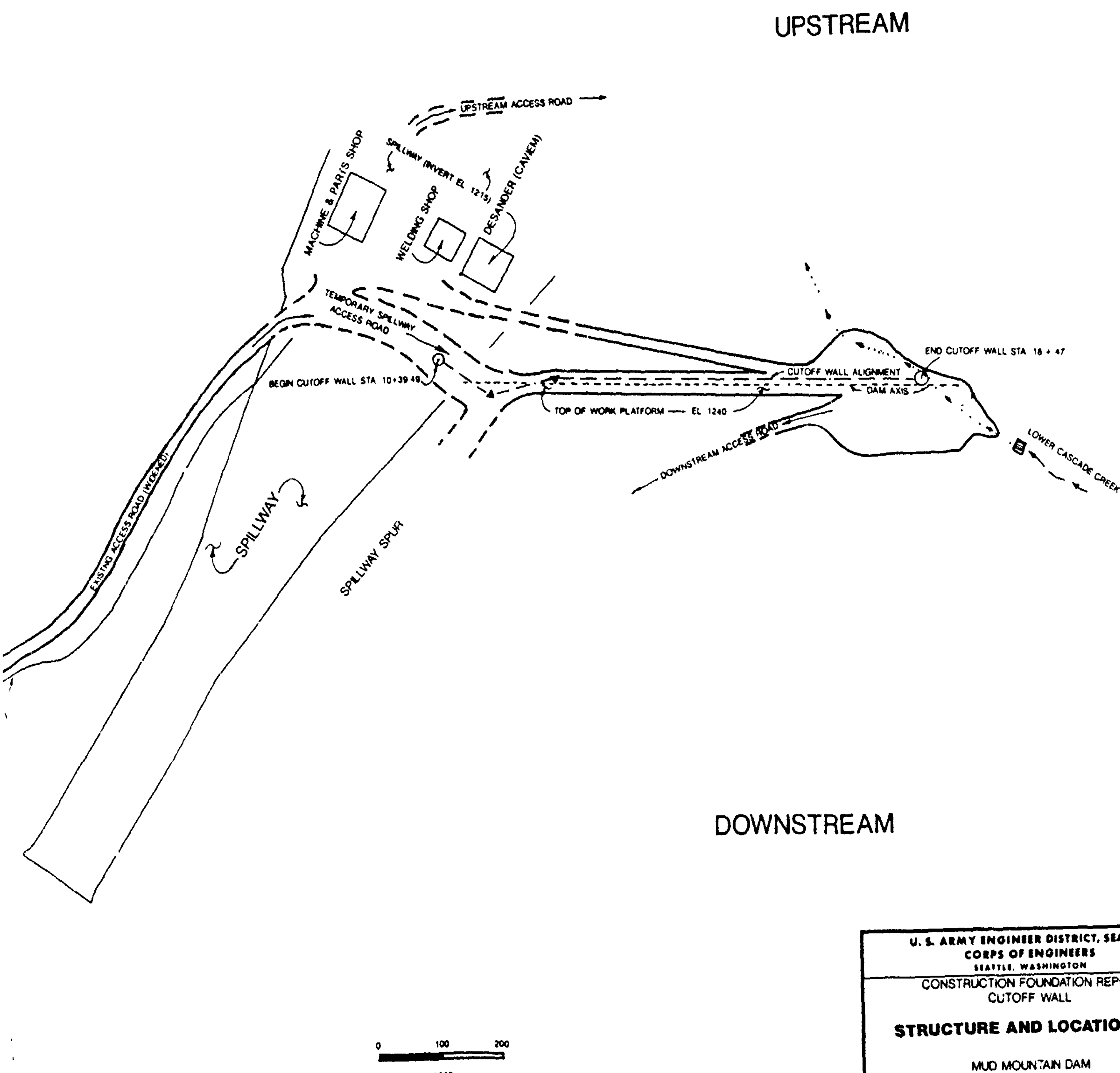
U. S. ARMY ENGINEER DISTRICT, SEATTLE **CORPS OF ENGINEERS** **SEATTLE, WASHINGTON**
CONSTRUCTION FOUNDATION REPORT CUTOFF WALL **TITLE AND AREA MAPS** MUD MOUNTAIN DAM

SIZE	INVITATION NO	FILE NO	DATE
DSGN SATTER		CHK	SHEET

3 | 2 | 1



5
4
3
D
C
B
A
N
GOVERNMENT PROPERTY BOUNDARY
TO HIGHWAY 410
WASTE AREA (BENTONITE)
HAUL ROAD
WASTE AREA (SAND & GRAVEL)
COE
RESIDENT OFFICE
CONCRETE
BATCH PLANT
SLURRY
MIXING PLANT
SOLETANCHE OFFICES
EXISTING ACCESS ROAD (WIDENED)
SLURRY LINES
(TO DESANDER)
SPILLWAY



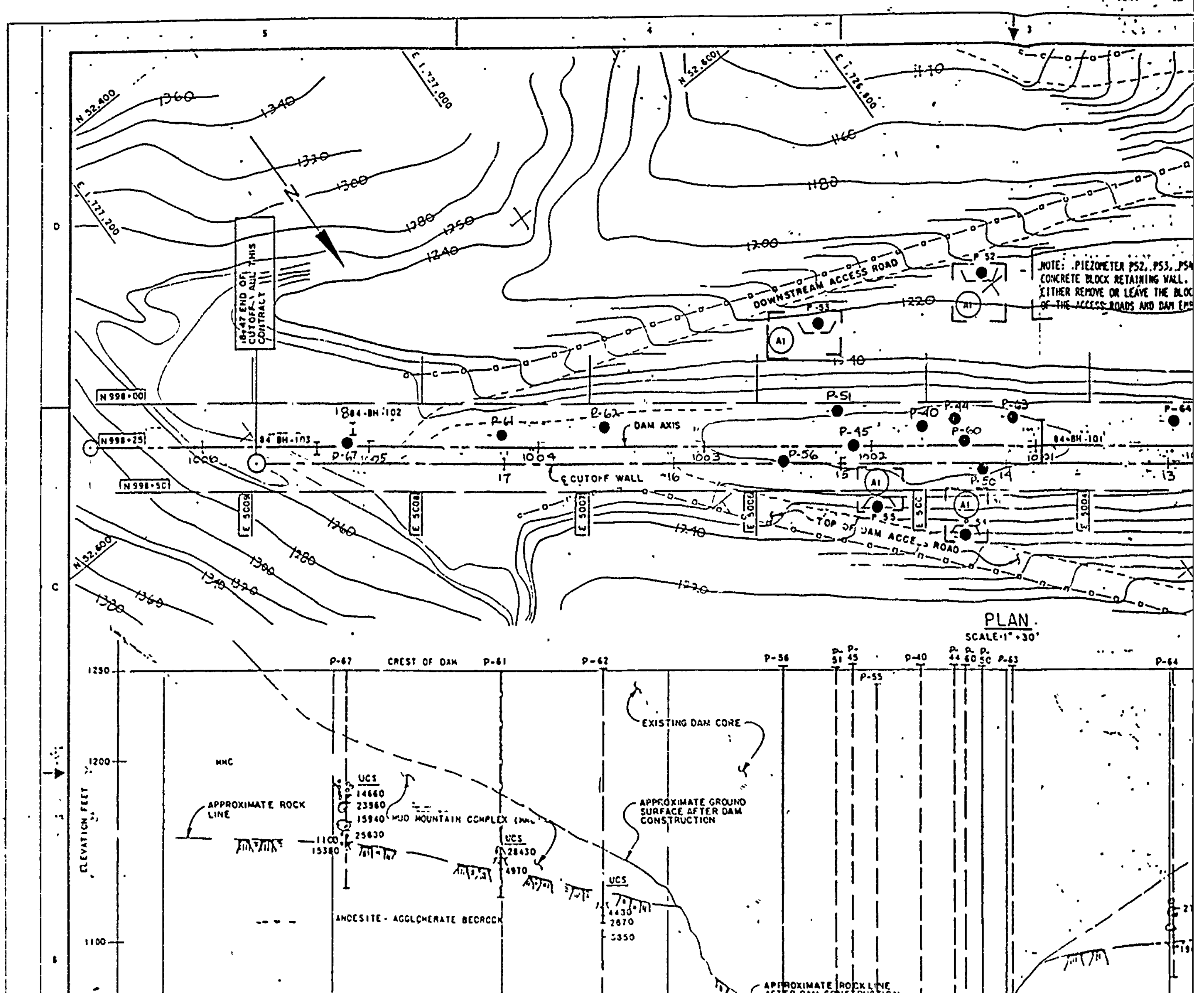
U. S. ARMY ENGINEER DISTRICT, SEATTLE
CORPS OF ENGINEERS
SEATTLE, WASHINGTON

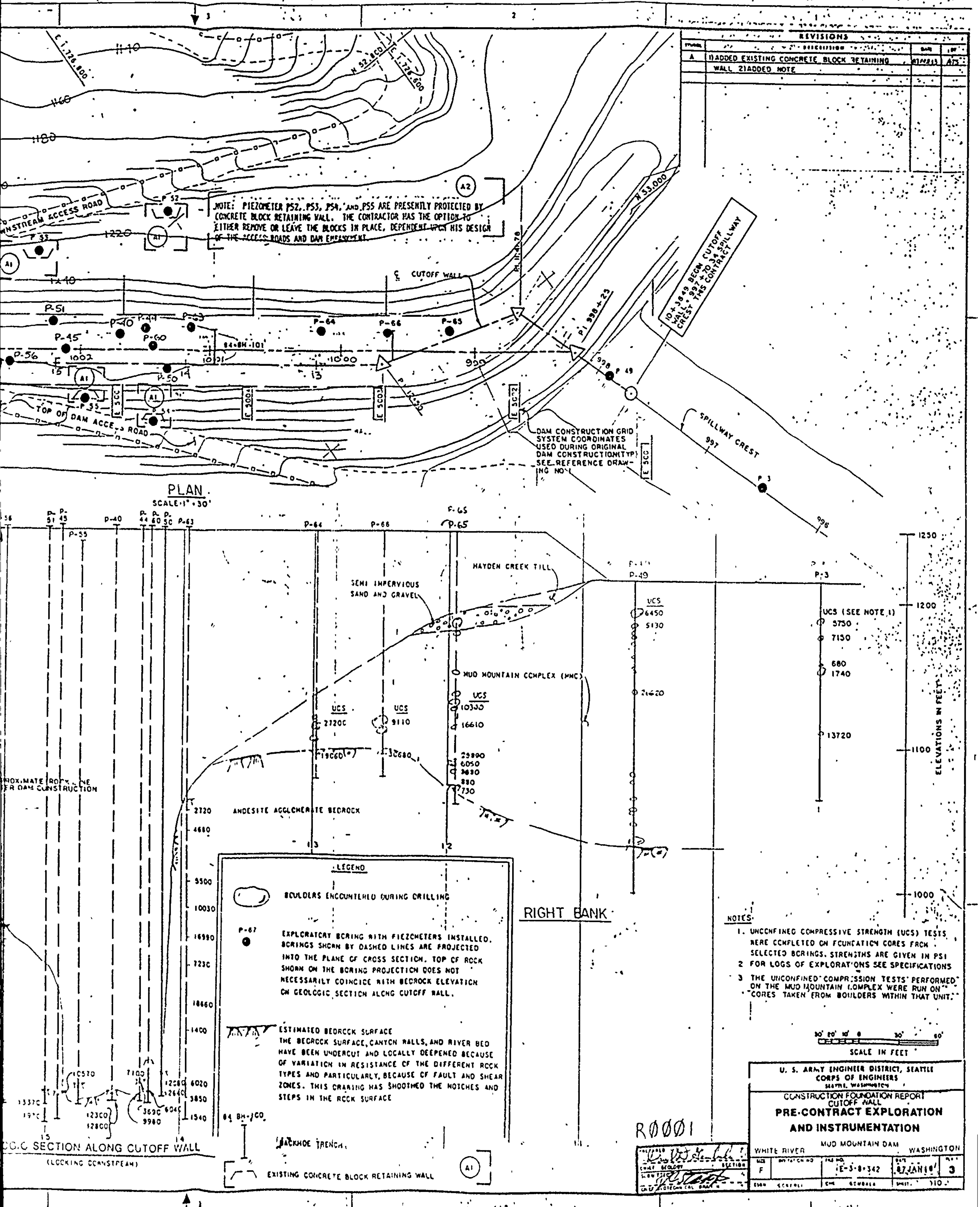
CONSTRUCTION FOUNDATION REPORT
CUTOFF WALL

STRUCTURE AND LOCATION MAP

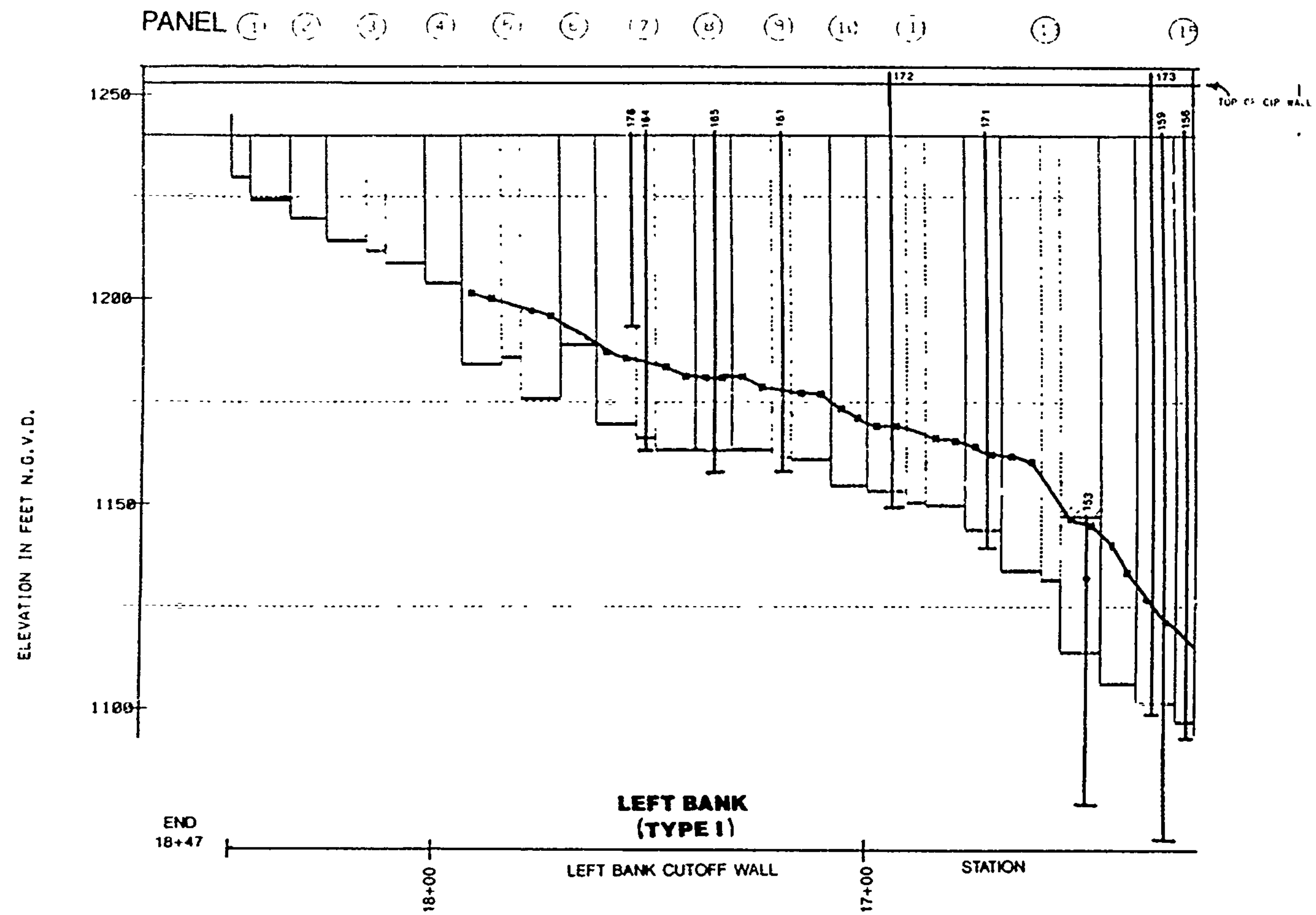
MUD MOUNTAIN DAM

SIZE	INVITATION NO	FILE NO	DATE	PLATE
				2
DSGN SATTER		CHK	SHEET	

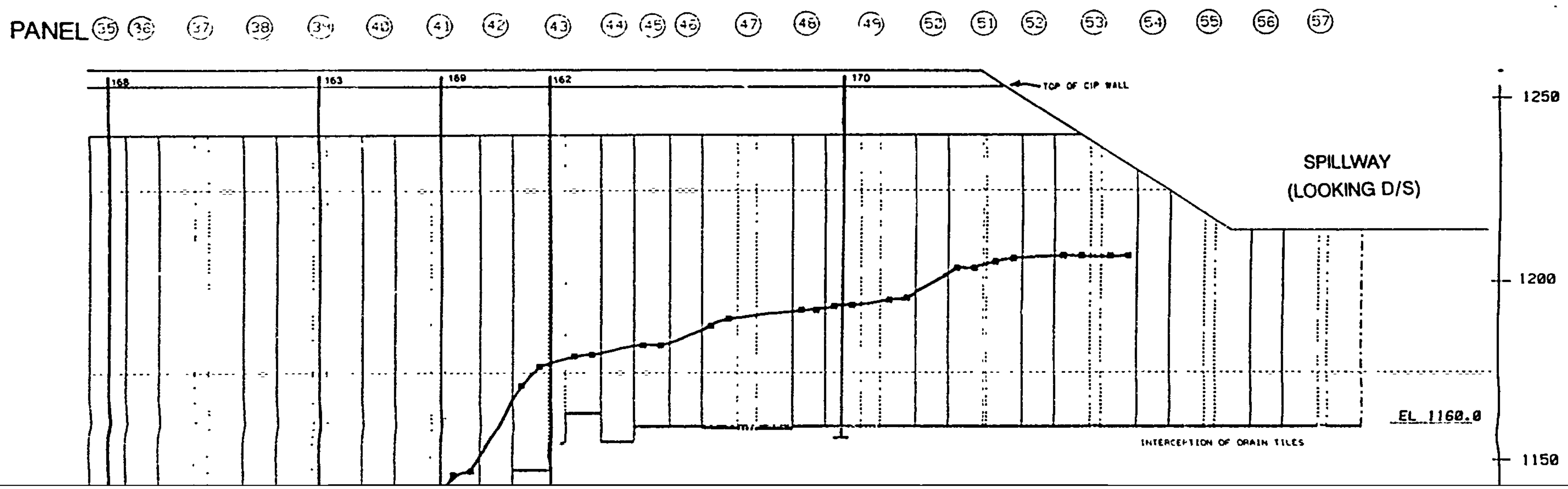

PLAN
SCALE 1" = 30'
DOWNSTREAM ACCESS ROAD
TOP OF DAM ACCESS ROAD
DAM AXIS
CUTOFF WALL
NOTE: PIEZOMETER P52, P53, P54
CONCRETE BLOCK RETAINING WALL.
EITHER REMOVE OR LEAVE THE BLOCK
OF THE ACCESS ROADS AND DAM EMB
END OF CUTOFF WALL THIS CONTRACT
CREST OF DAM
EXISTING DAM CORE
APPROXIMATE ROCK LINE
MUD MOUNTAIN COMPLEX
APPROXIMATE GROUND SURFACE AFTER DAM CONSTRUCTION
ANDESITE - AGGLOMERATE BEDROCK
APPROXIMATE ROCK LINE AFTER DAM CONSTRUCTION
ANDESITE AGGLOMERATE
UCS (SEE NOTE 1)
ELEVATION FEET
LEFT BANK
GEOLOGIC SECTION ALONG CUTOFF WALL
(LOOKING DOWNSTREAM)
BOULDERS
EXPLORATORY BORINGS SHOWN INTO THE SHOWN ON NECESSARILY ON GEOLOGIC
ESTIMATED THE BEDROCK HAVE BEEN OF VARIATIO TYPES AND ZONES. THIS STEPS IN TH
BACKHOE TR
EXISTING CONCR



REVISIONS
A 1) ADDED EXISTING CONCRETE BLOCK RETAINING WALL 2) ADDED NOTE
NOTE: PIEZOMETER P52, P53, P54, AND P55 ARE PRESENTLY PROTECTED BY CONCRETE BLOCK RETAINING WALL. THE CONTRACTOR HAS THE OPTION TO EITHER REMOVE OR LEAVE THE BLOCKS IN PLACE, DEPENDENT UPON HIS DESIGN OF THE ACCESS ROADS AND DAM EMBANKMENT.
DOWNSTREAM ACCESS ROAD
TOP OF DAM ACCESS ROAD
℄ CUTOFF WALL
10+38.49 BEGIN CUTOFF WALL = 997+70.34 SPILLWAY CREST THIS CONTRACT
SPILLWAY CREST
DAM CONSTRUCTION GRID SYSTEM COORDINATES USED DURING ORIGINAL DAM CONSTRUCTION (TYP) SEE REFERENCE DRAWING NO. 1
PLAN
SCALE 1" = 30'
HAYDEN CREEK TILL
SEMI IMPERVIOUS SAND AND GRAVEL
MUD MOUNTAIN COMPLEX (MMC)
ANDESITE AGGLOMERATE BEDROCK
APPROXIMATE ROCK LINE AFTER DAM CONSTRUCTION
UCS (SEE NOTE 1)
ELEVATIONS IN FEET
RIGHT BANK
GEOLOGIC SECTION ALONG CUTOFF WALL
(LOOKING DOWNSTREAM)
LEGEND
BOULDERS ENCOUNTERED DURING DRILLING
P-67
EXPLORATORY BORING WITH PIEZOMETERS INSTALLED. BORINGS SHOWN BY DASHED LINES ARE PROJECTED INTO THE PLANE OF CROSS SECTION. TOP OF ROCK SHOWN ON THE BORING PROJECTION DOES NOT NECESSARILY COINCIDE WITH BEDROCK ELEVATION ON GEOLOGIC SECTION ALONG CUTOFF WALL.
ESTIMATED BEDROCK SURFACE
THE BEDROCK SURFACE, CANYON WALLS, AND RIVER BED HAVE BEEN UNDERCUT AND LOCALLY DEEPENED BECAUSE OF VARIATION IN RESISTANCE OF THE DIFFERENT ROCK TYPES AND PARTICULARLY, BECAUSE OF FAULT AND SHEAR ZONES. THIS DRAWING HAS SMOOTHED THE NOTCHES AND STEPS IN THE ROCK SURFACE
84 BH-100
BACKHOE TRENCH
EXISTING CONCRETE BLOCK RETAINING WALL
NOTES:
1. UNCONFINED COMPRESSIVE STRENGTH (UCS) TESTS WERE COMPLETED ON FOUNDATION CORES FROM SELECTED BORINGS. STRENGTHS ARE GIVEN IN PSI
2. FOR LOGS OF EXPLORATIONS SEE SPECIFICATIONS
3. THE UNCONFINED COMPRESSION TESTS PERFORMED ON THE MUD MOUNTAIN COMPLEX WERE RUN ON CORES TAKEN FROM BOULDERS WITHIN THAT UNIT.
SCALE IN FEET
U. S. ARMY ENGINEER DISTRICT, SEATTLE
CORPS OF ENGINEERS
SEATTLE, WASHINGTON
CONSTRUCTION FOUNDATION REPORT
CUTOFF WALL
PRE-CONTRACT EXPLORATION AND INSTRUMENTATION
MUD MOUNTAIN DAM
WHITE RIVER WASHINGTON
R0001

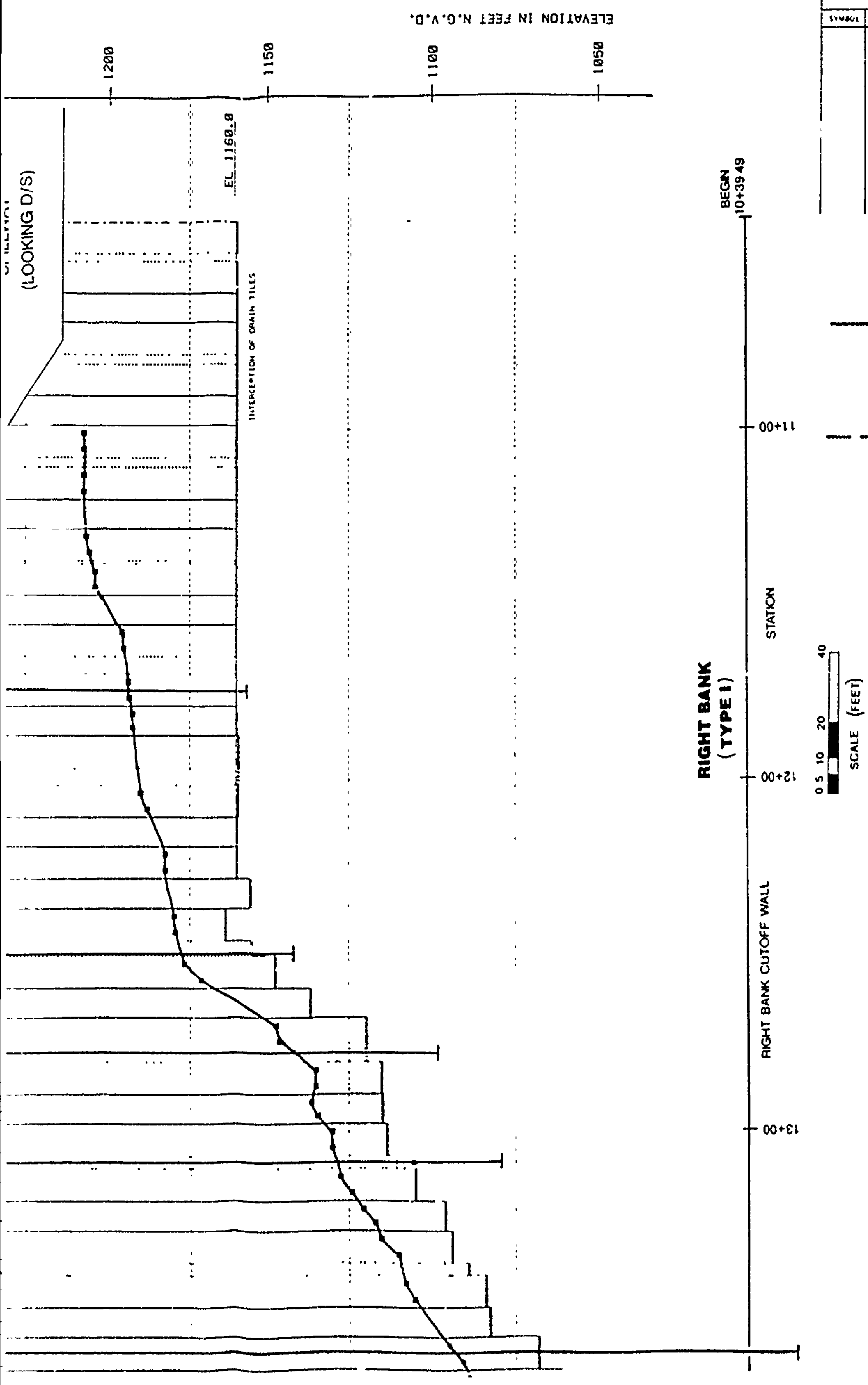


PANEL 1 2 3 4 5 6 7 8 9 10 11 13
1250
1200
1150
1100
ELEVATION IN FEET N.G.V.D.
TOP OF CIP WALL
172
173
176
184
185
161
171
159
156
153
LEFT BANK
(TYPE I)
END
18+47
18+00
17+00
LEFT BANK CUTOFF WALL
STATION



PANEL 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57
168
163
169
162
170
TOP OF CIP WALL
SPILLWAY
(LOOKING D/S)
1250
1200
1150
EL 1160.0
INTERCEPTION OF DRAIN TILES

REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY



CONCRETE QC/EXPLORATORY HOLES (151 - 199)

NOTE: INCLINOMETERS INSTALLED IN # 178,182,183, 187,190,196 AND 199(D/S). HOLES LEFT OPEN IN #155,158,162,163,168,169,170,172,180 AND 198.

PIEZOMETERS (101 - 107) (108 - 111 SHOWN ON FIG X - 1).

PANEL NUMBERS SHOWN FOR CUTOFF WALL ONLY.

C.I.P. WALL NUMBERS SHOWN ON PLATE 6.

HOLE DEPTHS IN TABLES III-1 AND III-2.

PANEL DEPTHS IN TABLES XI-2A,B AND C

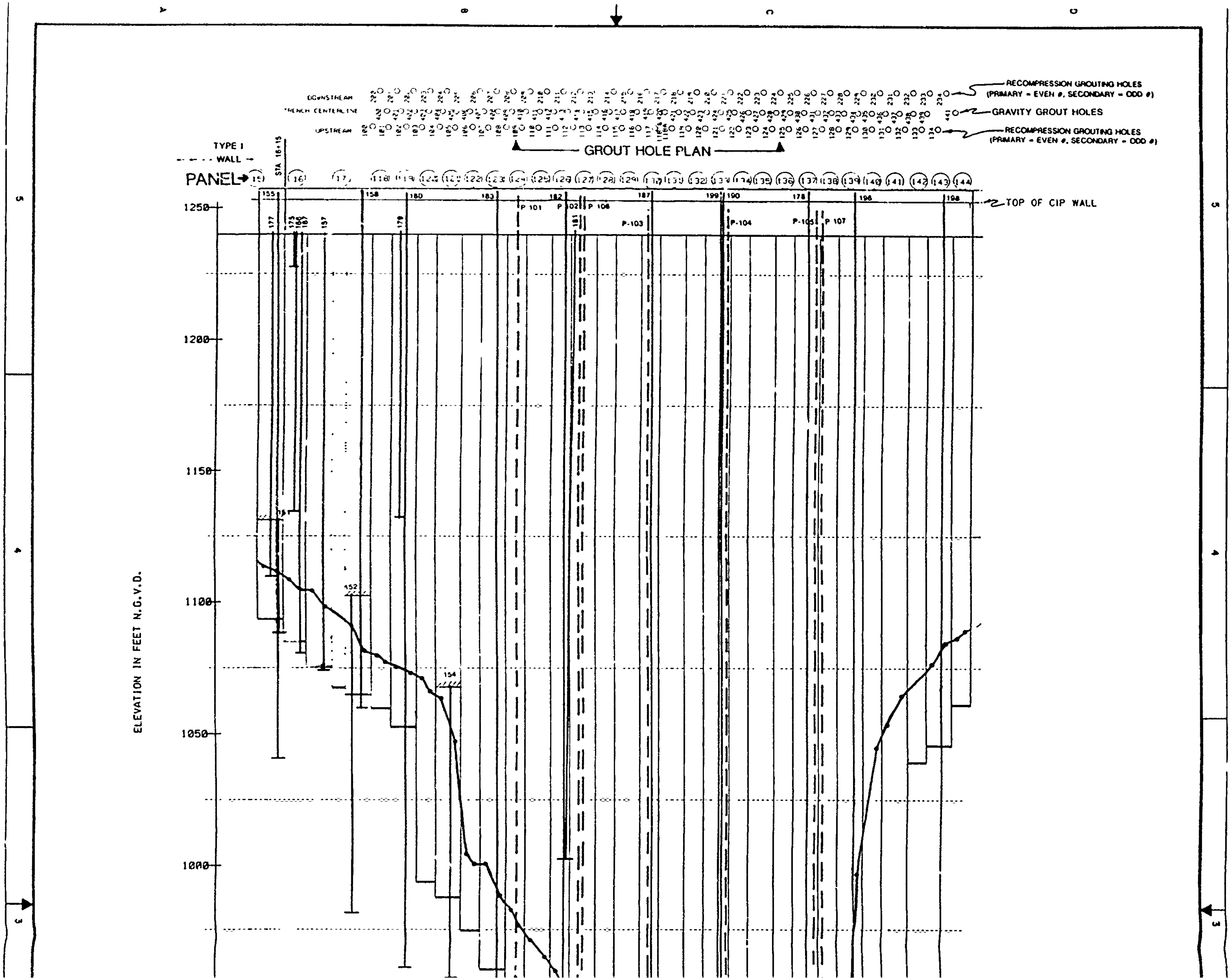
U. S. ARMY ENGINEER DISTRICT, SEATTLE
CORPS OF ENGINEERS
SEATTLE, WASHINGTON

CONSTRUCTION FOUNDATION REPORT
CUTOFF WALL

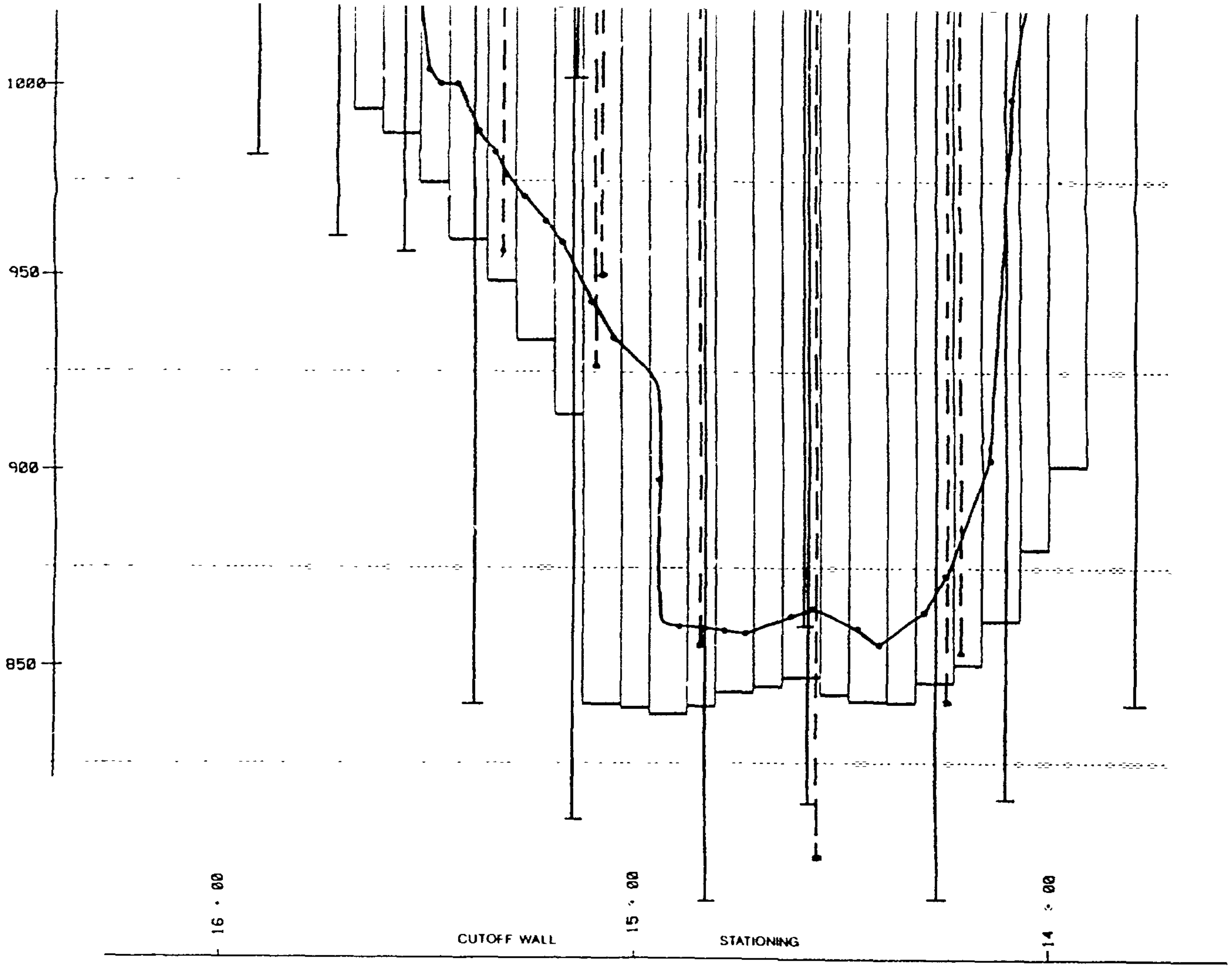
CUTOFF WALL (TYPE I)

MUD MOUNTAIN DAM

SIZE	INVITATION NO	FILE NO	DATE	PLATE
				4
DRWN SATTER		CHK	SHEET	



RECOMPRESSION GROUTING HOLES (PRIMARY = EVEN #, SECONDARY = ODD #)
GRAVITY GROUT HOLES
RECOMPRESSION GROUTING HOLES (PRIMARY = EVEN #, SECONDARY = ODD #)
DOWNSTREAM
TRENCH CENTERLINE
UPSTREAM
GROUT HOLE PLAN
TYPE I WALL
PANEL
STA 16+15
TOP OF CIP WALL
P 101
P 102
P 106
P-103
P-104
P-105
P 107
1250
1200
1150
1100
1050
1000
ELEVATION IN FEET N.G.V.D.



CANYON SECTION — TYPE II

(LOOKING DOWNSTREAM)

0 5 10 20 40
SCALE (FEET)

CONCRETE QC/EXPLORATORY HOLES (151 - 199)

INCLINOMETERS INSTALLED IN # 178, 182 183 187, 190, 196 AND 199(D/S)
HOLES LEFT OPEN IN # 155, 158, 162, 163, 168 169, 170, 172, 180 AND 198)

PIEZOMETERS (101 - 107). (108 111 SHOWN ON FIG X - 1).

PANEL NUMBERS SHOWN FOR CUTOFF WALL ONLY. C.I.P WALL NUMBERS SHOWN ON PLATE 6.

HOLE DEPTHS IN TABLES III-1 AND III-2 PANEL DEPTHS IN TABLES XI-2A,B AND C.

REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY

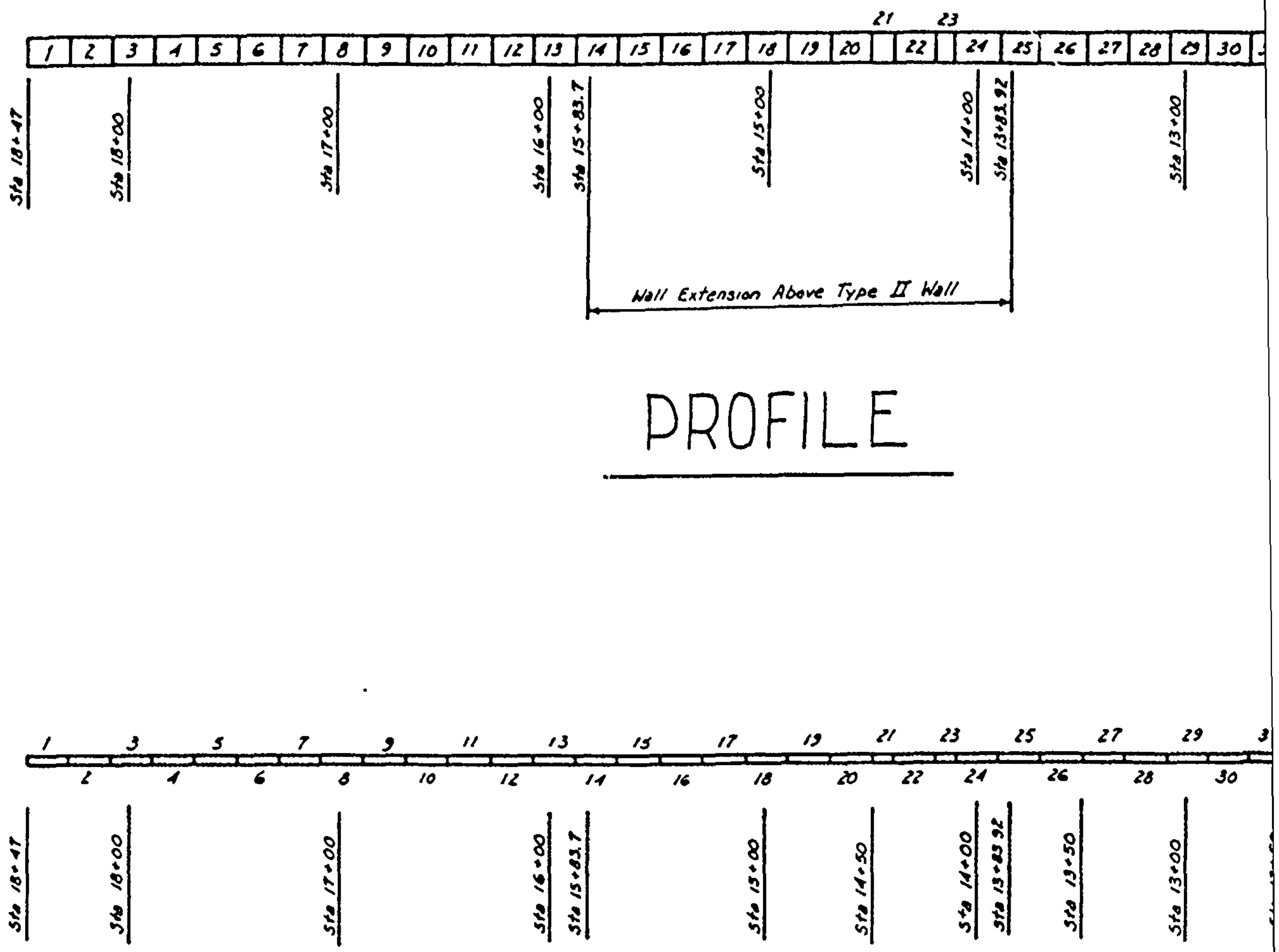
U. S. ARMY ENGINEER DISTRICT, SEATTLE
CORPS OF ENGINEERS
SEATTLE WASHINGTON

CONSTRUCTION FOUNDATION REPORT
CUTOFF WALL

CUTOFF WALL (TYPE II)

MUD MUD MOUNTAIN DAM

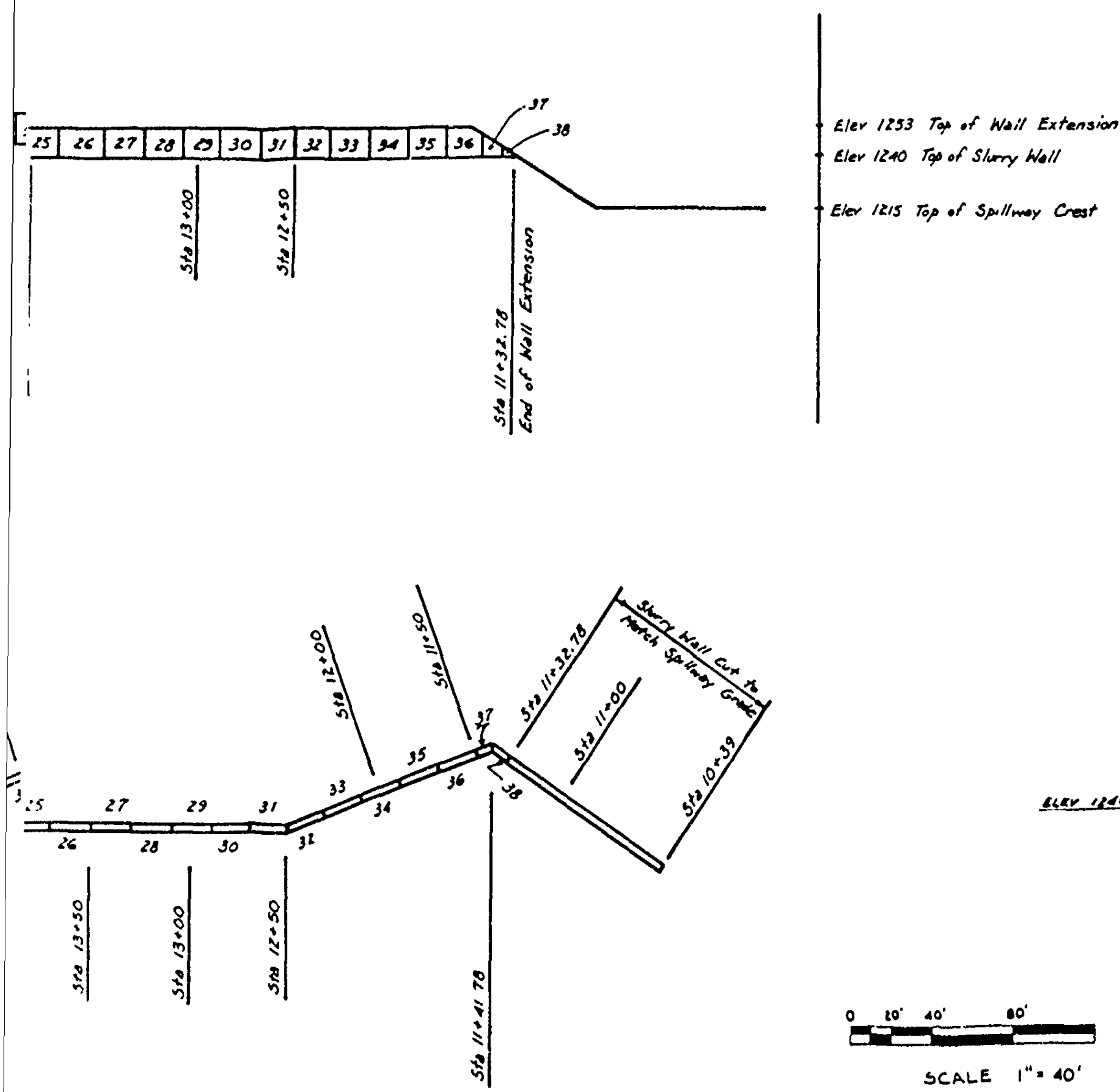
SIZE	INVITATION NO	FILE NO	DATE	PLATE 5
DSGN SATTER		CHK	SHEET	

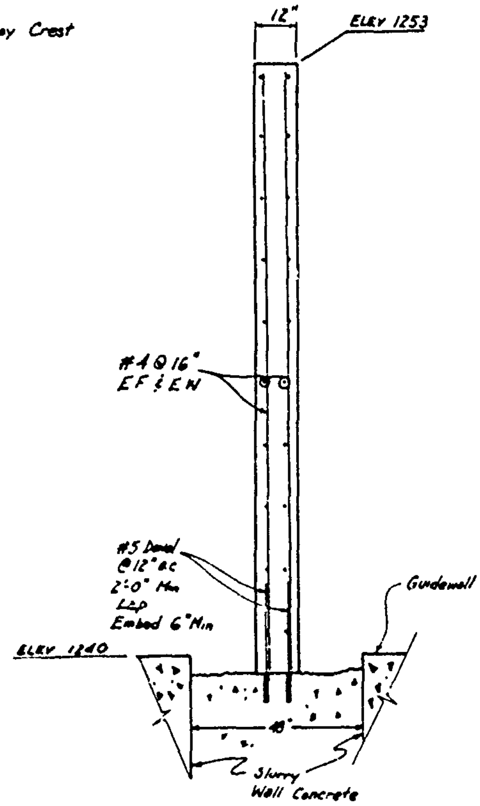


PROFILE

PLAN

REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY





TYPICAL SECTION

NTS

0 20' 40' 80'

SCALE 1" = 40'

U. S. ARMY ENGINEER DISTRICT, SEATTLE
CORPS OF ENGINEERS
SEATTLE, WASHINGTON

CONSTRUCTION FOUNDATION REPORT
CUTOFF WALL

CAST-IN-PLACE (C.I.P.)

CUTOFF WALL EXTENSION

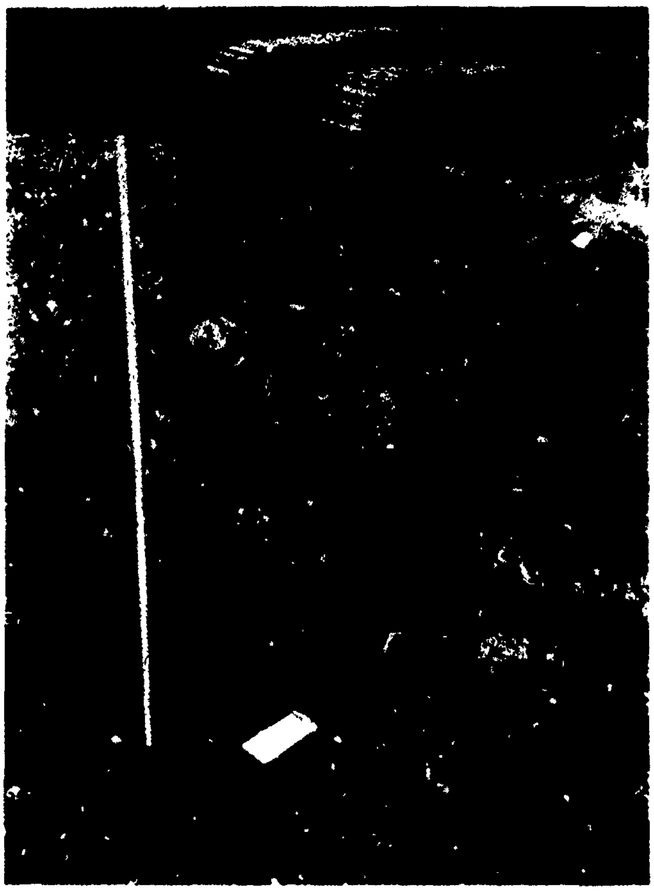
MUD MOUNTAIN DAM

SIZE	INVITATION NO	FILE NO	DATE	PLATE
				6
DSGN		CHK	SHEET	

PHOTOGRAPHS

Spillway slab is pre-cut for this portion of cutoff wall. Access road fill built up over this area.

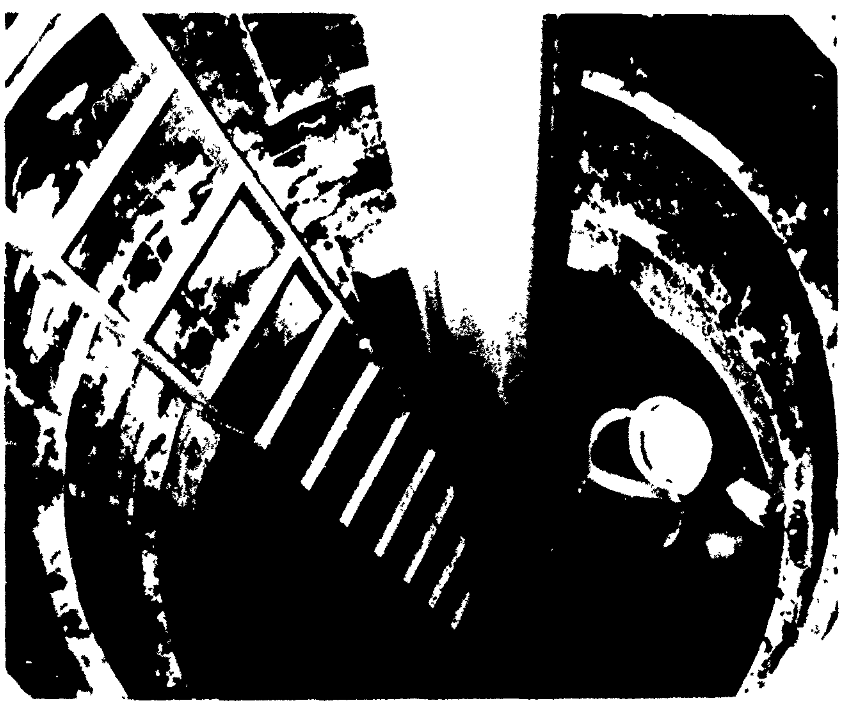
Access road built up over spillway. Existing access road in foreground. Lowered dam at upper right. Upstream to the left.



During dam lowering from El. 1250 to El. 1240, several weathered longitudinal cracks were found. Rock hammer for scale.

Geotextile placement on lowered dam work platform. 6" of base course material over fabric.

6 ft. diameter auger removing core material from access shaft.

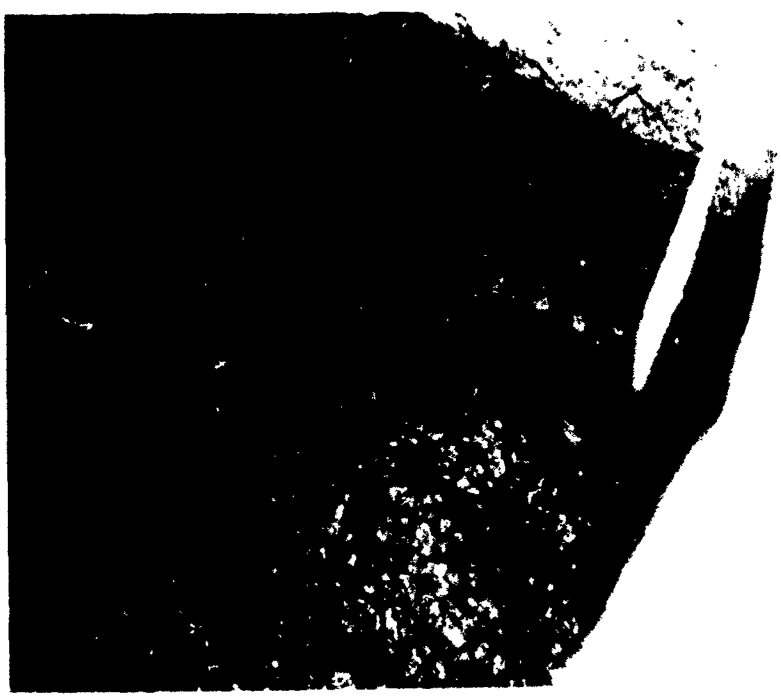


Inspecting and sampling core material at 7 ft. depth. Note auger is below, Kelly bar is center.

Inspector being lowered into access shaft with a man basket. Note liner plates, left.

Density test being taken in core material, 84 ft. depth.

P-5Ø 6" casing in access shaft @ 1Ø7 ft. depth. Note perforations did not penetrate casing.



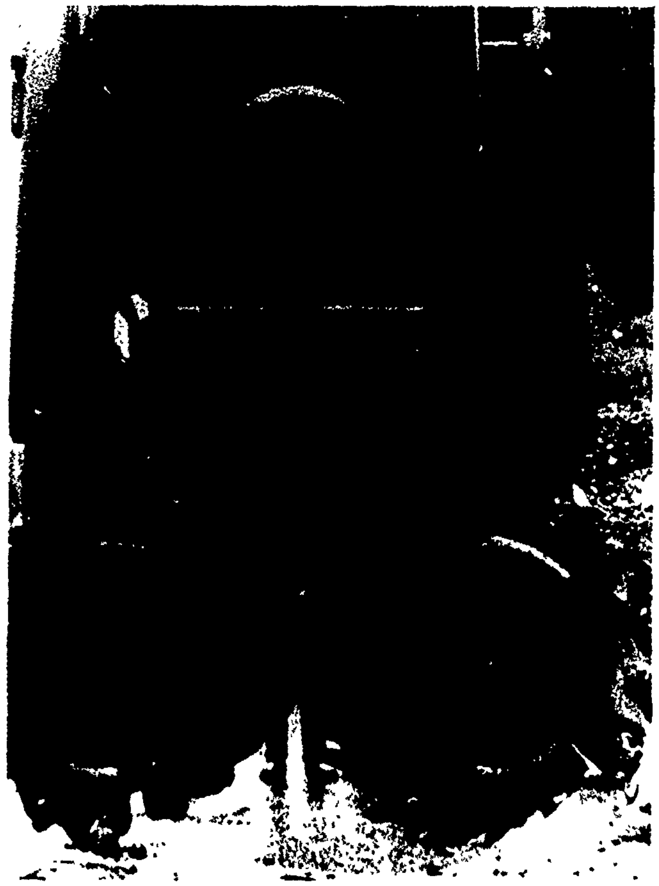
Water seeping into access shaft from core fracture at 136 ft. depth.

Typical construction of guide walls for hydrofraise.

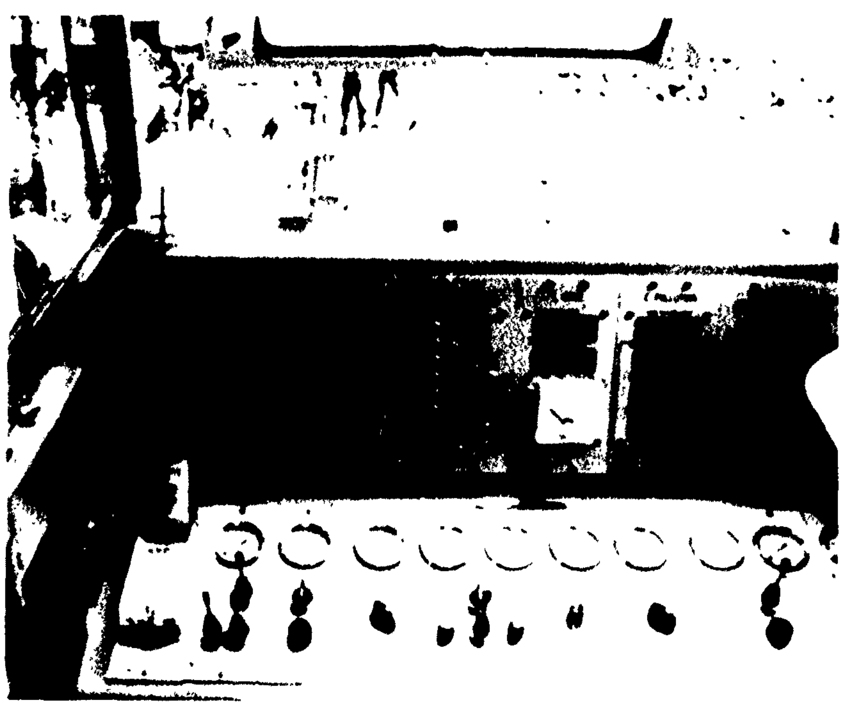
One of four bentonite slurry ponds. Note central pump and agitator. Concrete plant in background.

Looking at right side of spillway invert. Desander (Caviem) at right, welding shop in middle and machine/parts shop in background. Note spillway access road at left.

Business end of Hydrofraise. Finishing a "pic" (tooth) change and ready to go back into panel (left). Note the guide frame on top of wall.

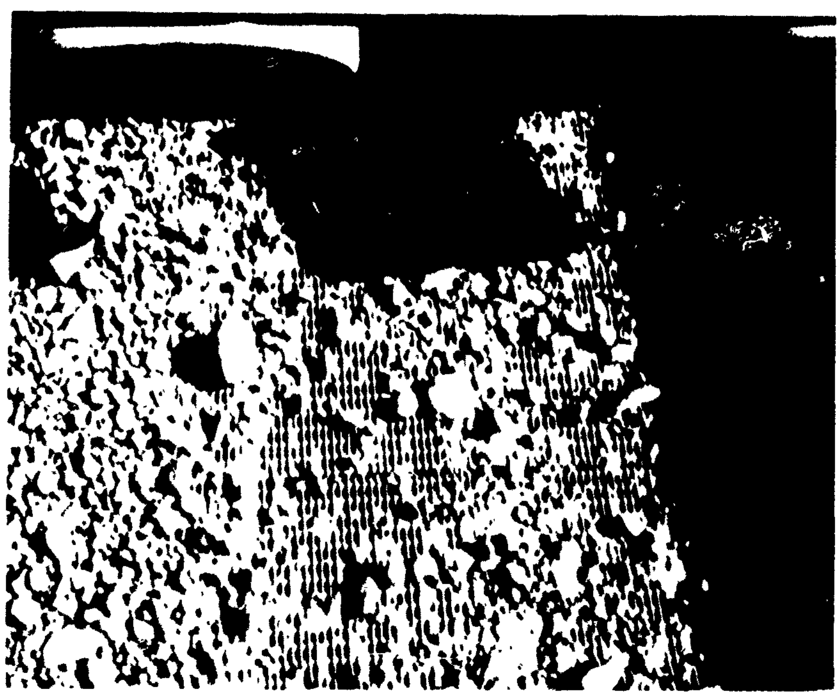


Side view of hydrofraise. Drum at left rotates counter clock wise while right turns opposite direction. Partial view of huge suction pump at middle, top.

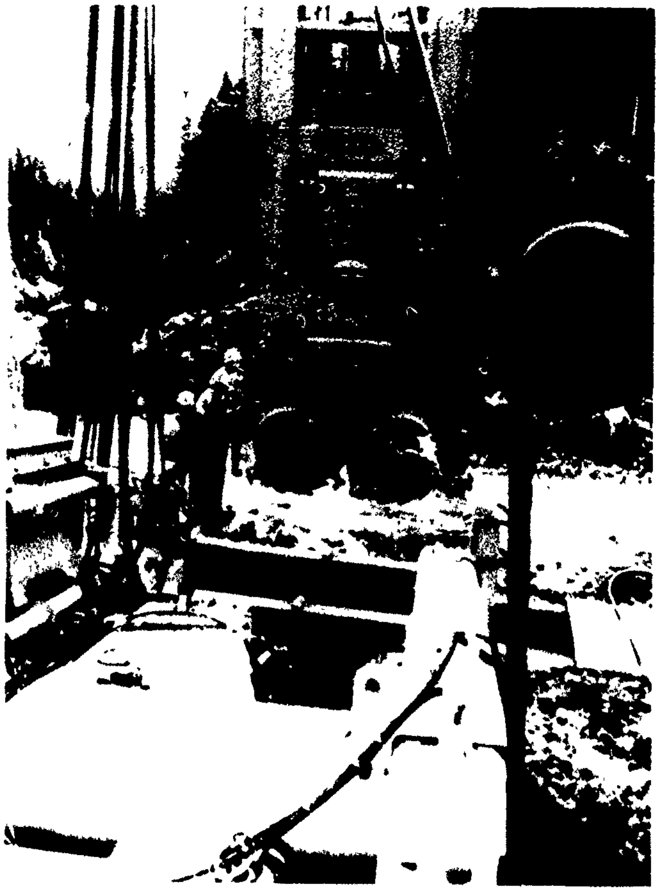


View from hydrofraise operations cab. 4100 crane operator's cab is immediately to the left. Hydrofraise at top left.

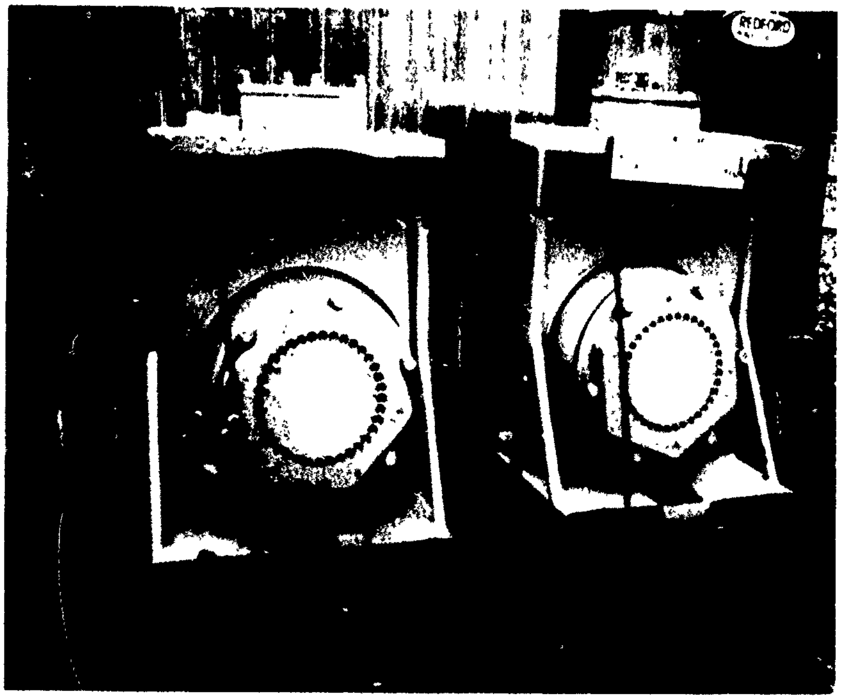
Hydrofraise excavating Type I panel in left back. Looking down stream. Top portion of hydrofraise frame is visible. Cutter heads at 75 ft. depth . Note guide frame on top of wall.



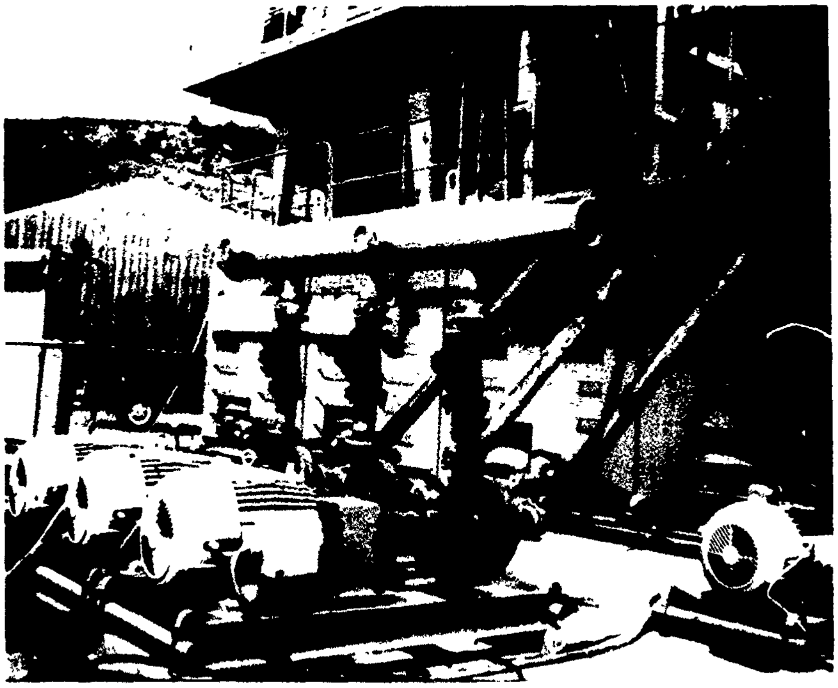
Coarse sand and gravel faction of excavated core material separated out on one of two screens of desander. Material vibrated off the side of desander into spoils pile which is transported to waste area.



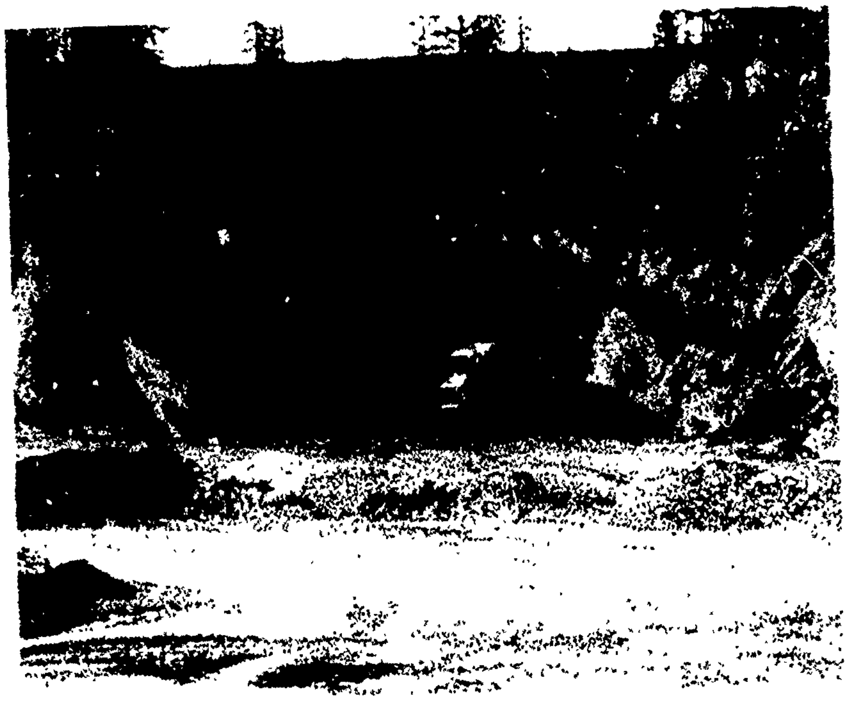
View of hydrofraise from operator's cab.



Spare hydrofraise motors.



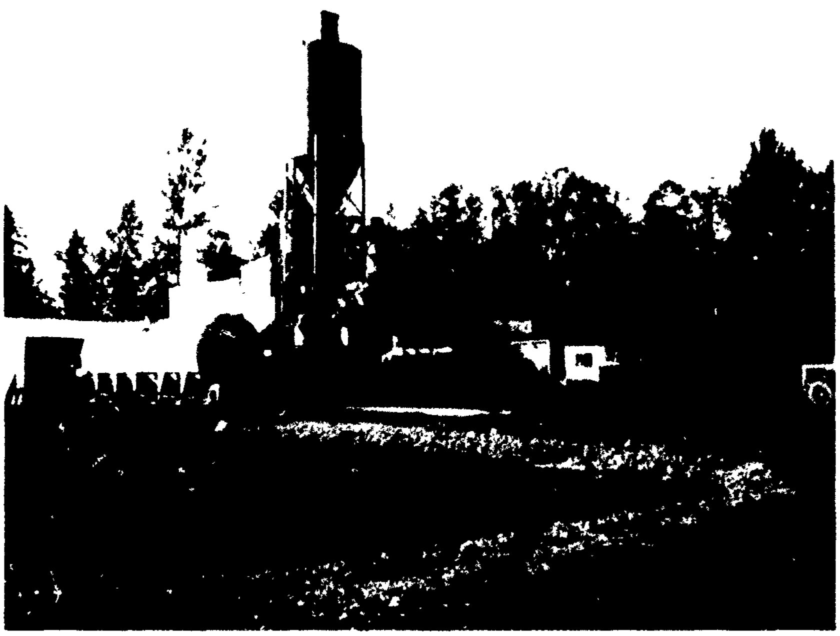
Three Mission pumps outside desander that pump recycled slurry back to hydrofraise. Slurry holding tanks below, desander portion above.



Cadman pit showing aggregate materials source.

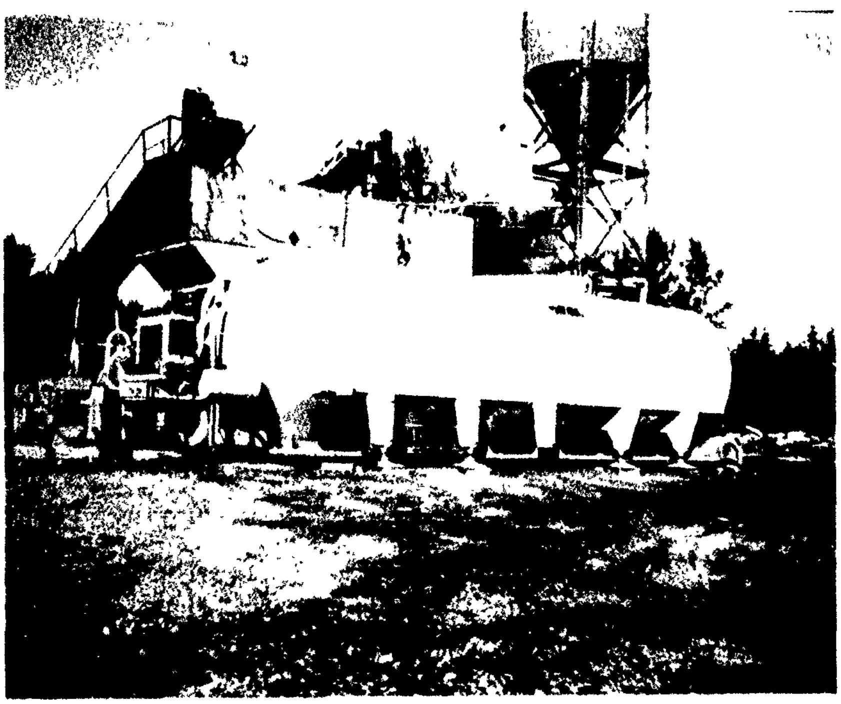
Cadman wash plant with settling pond in foreground.

Cadman wash plant arranged to produce 1 1/2-inch, 7/8-inch, 3/8-inch nominal maximum sized aggregates and sand.



Batch plant with 115 tons cement silo, 100 tons pozzolan silo, and 175 tons storage guppy.

30-inch wide conveyors. Sand conveyor on the left and the coarse aggregate conveyor on the right. The contractor quality control laboratory is the building on the far left.



Batch plant with the 175 tons storage guppy in the foreground.

Winterization. 10-inch diameter steel pipe arrangement for the coarse aggregate stockpile.

Winterization. Steel pipe arranged for the sand stockpile. The building is the contractor quality control laboratory.

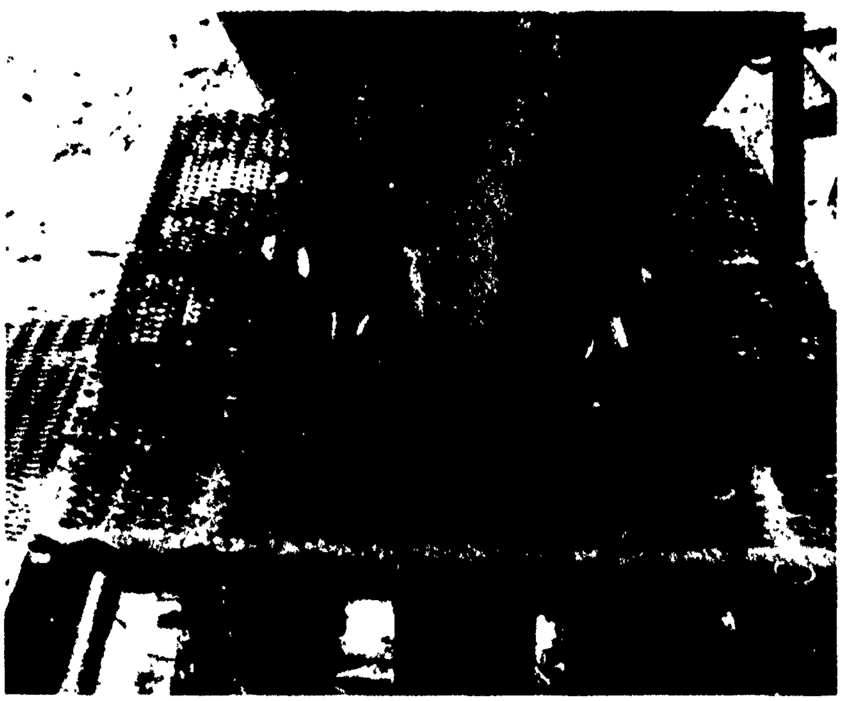
Winterization. Propane heaters in use on the coarse aggregate stockpile.

Winterization. Coarse aggregate stockpile showing tarpulin cover.

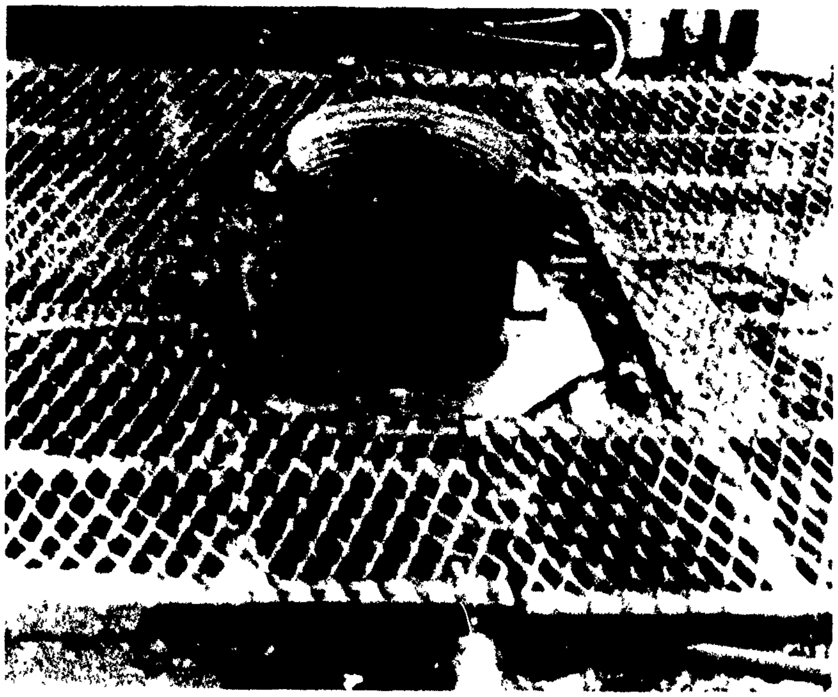
Winterization. Tarpulin cover to retain heat and keep precipatation off stockpile.

Winterization. Admixture storage tanks enclosed to keep from freezing.

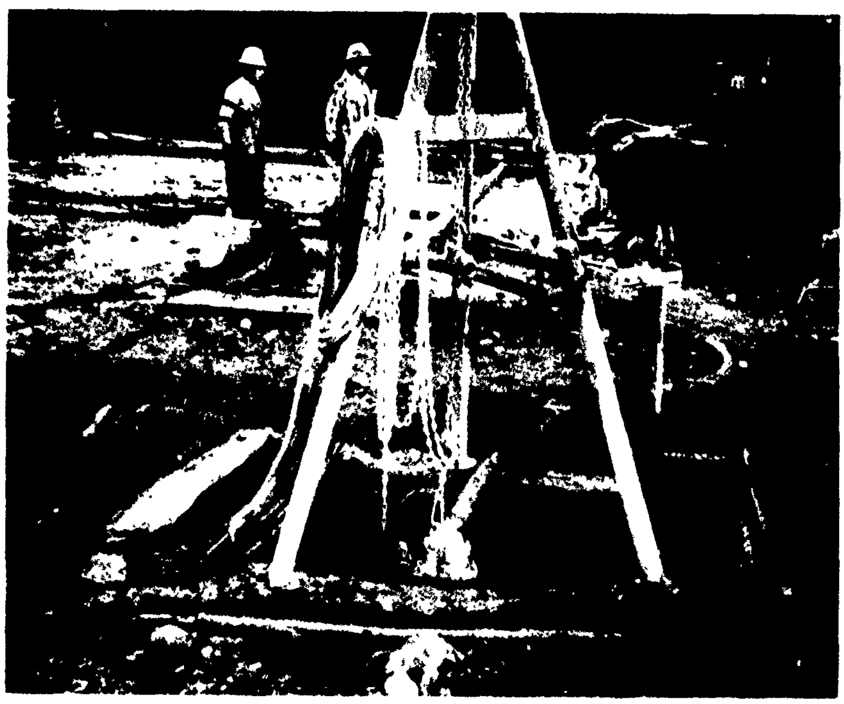
Winterization. The aggregate bins were roofed to keep precipatation from collecting in them.



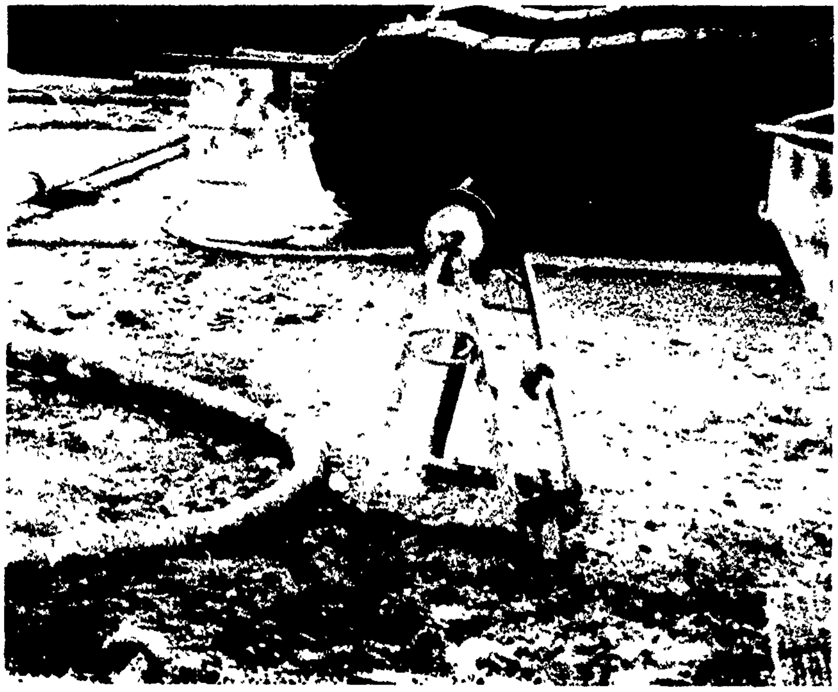
Funnel-shaped hopper resting on work platform.



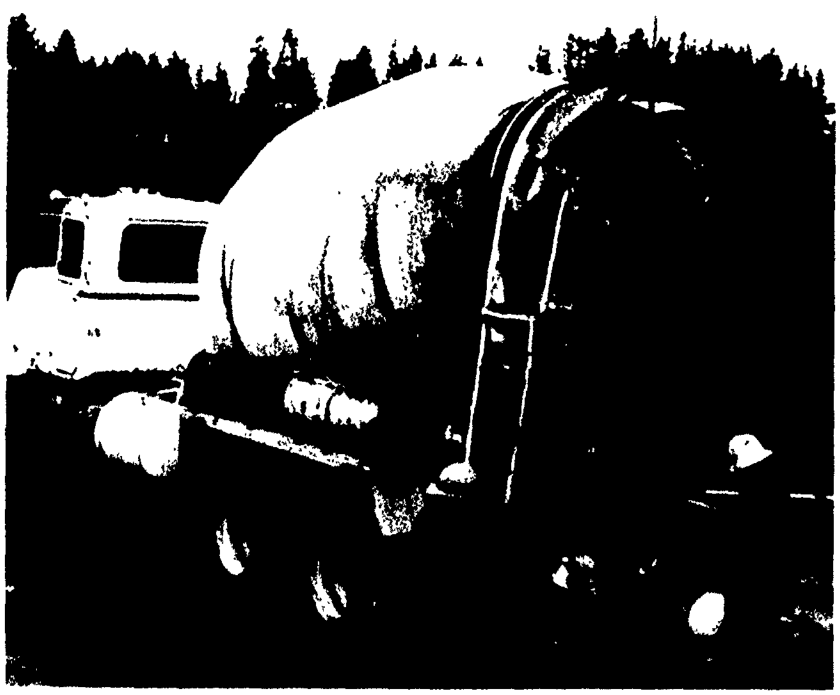
Tremie pipe coupling held in place with "dog ear" support plates.



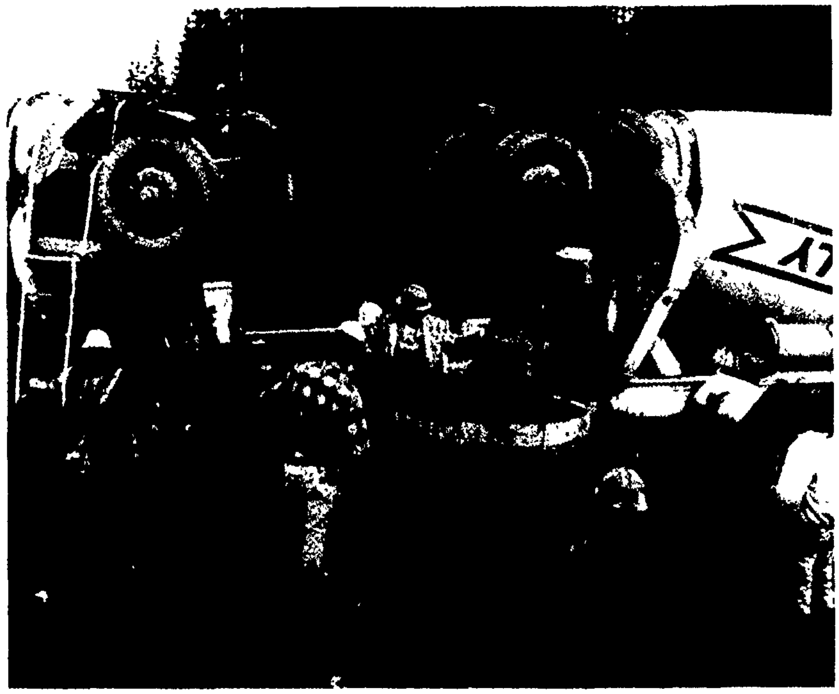
Submergible pump used to remove displaced bentonite slurry.



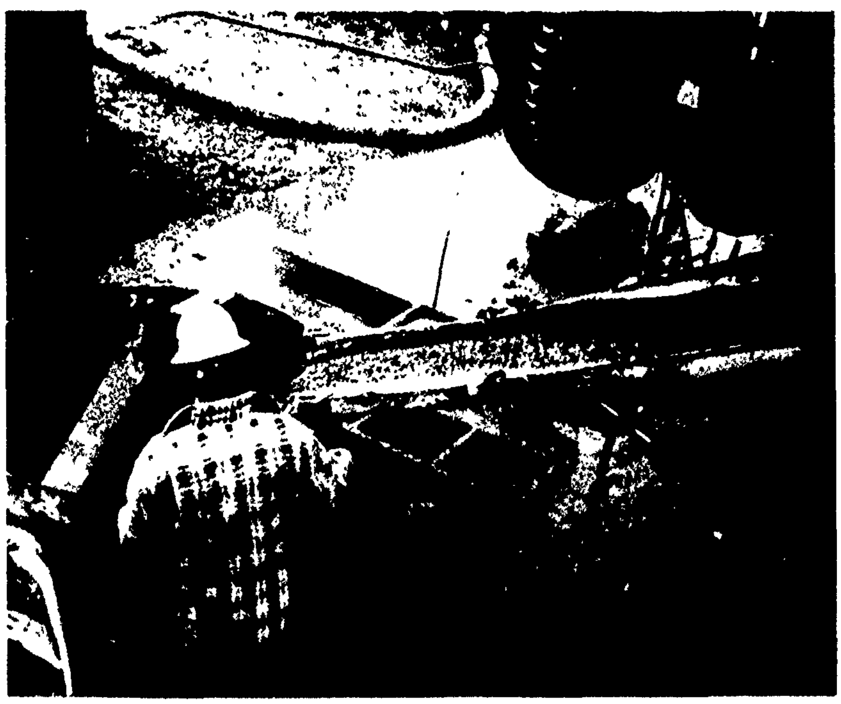
Sounding device used to determine the concrete surface depth beneath the slurry.



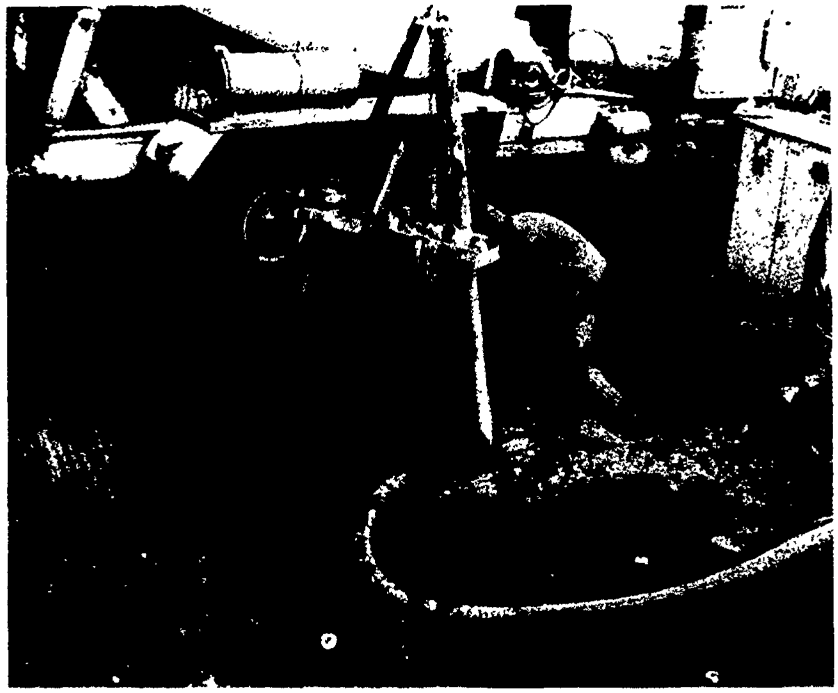
Typical 8-10 cubic yard transit mixer.



Transit mixers backed up to two tremies delivering concrete into one panel.

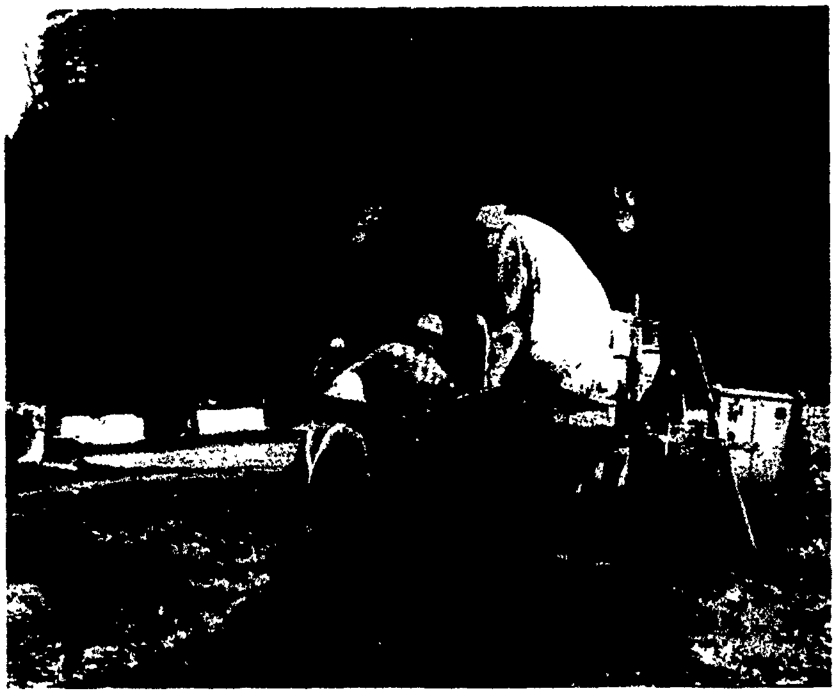


Flowable tremie concrete delivery into the 1.1 cy hopper.



Slurry overflow from the trench upon fast delivery of concrete.

Unscrewing hopper from pipe coupling to prepare for the shortening of the tremie pipe length.

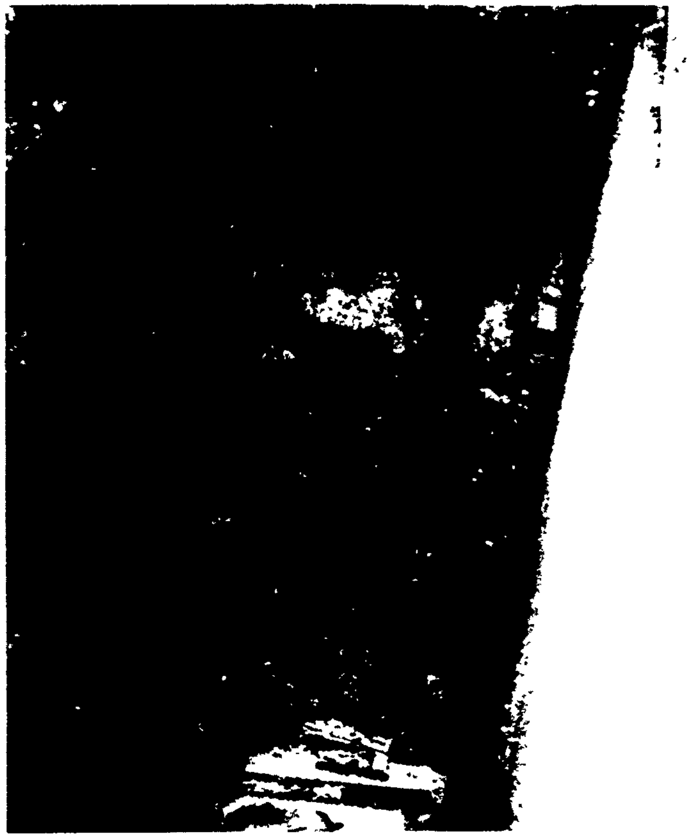


Removing one section of tremie pipe.

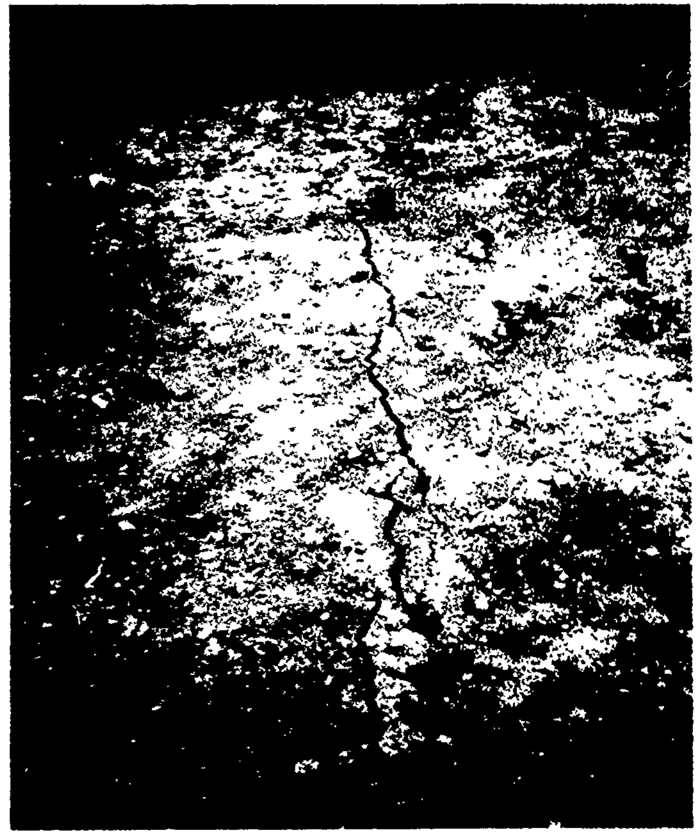
Laborer using chain wrench to unscrew pipe from coupling. Service crane is in the background.

Coring "fish" viewed from top. Two of four drill pipes visible. Pipes radius outward then cross over at bottom. Cores .9 inch . Instrumental in early detection of panel verticality error.

Business end of coring "fish." Drill rods exit the fish between spacer flanges, next to workman. Note lateral jacking pads which lock fish in place.



First indication of hydrofracture in Panel 35. Originally thought to be deeper expression of weathered surface cracks noted on surface during dam lowering. 200,000 gallons of slurry were lost before stabilizing panel. Panel is 40 inches wide. Looking at left wall. Upstream guidewall to left.



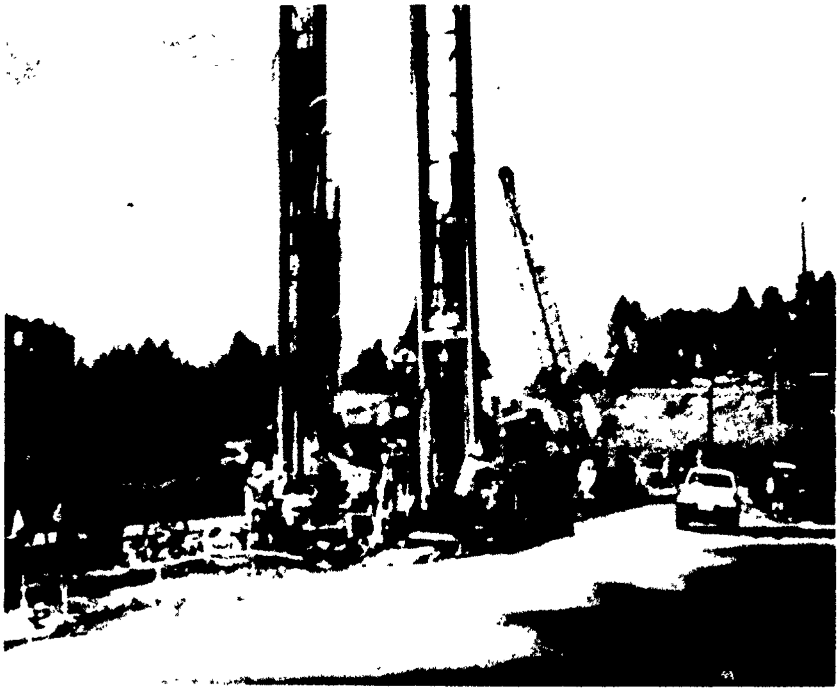
Cracks on work platform running longitudinally across dam. Micro-cassette recorder for scale. Looking toward left side, cutoff wall about 10 ft. left. Left side is dropping down. Looking at area around station 14+50.

Close-up of crack. Left side down dropped. These cracks spanned the deep section of the dam (Panel 17 to Panel 143) in several places.

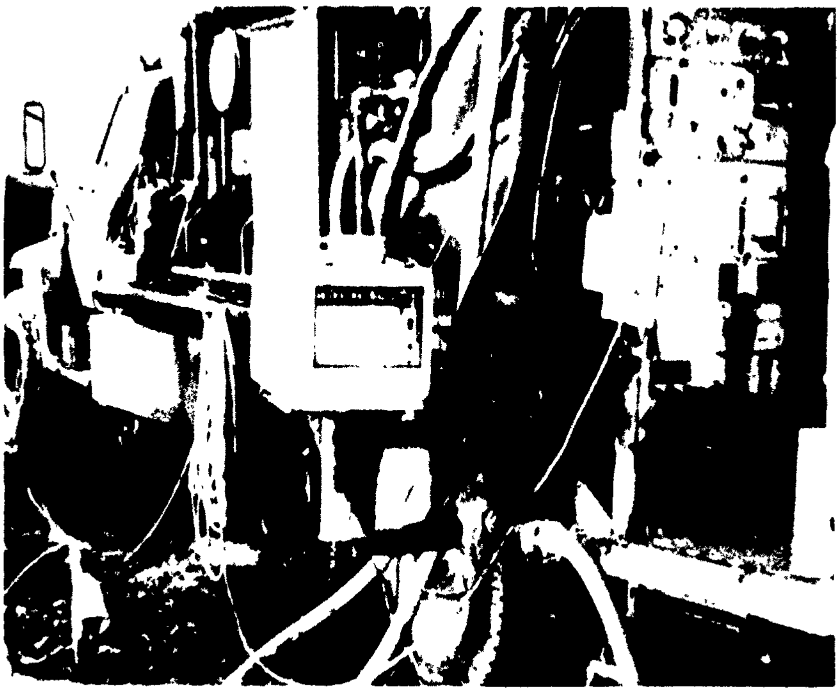
Hydrofracture crack in left wall of Panel 143. Panel is 40 inches wide. Note upstream (left) and downstream (right) guidewalls.

Hydrofracture in Panel 145 (?). Looking at left wall.

Emergency measures for uncontrolled slurry loss being implemented. Contractor is dumping wasted sand/silt mix from desander back into panel to help seal cracks.

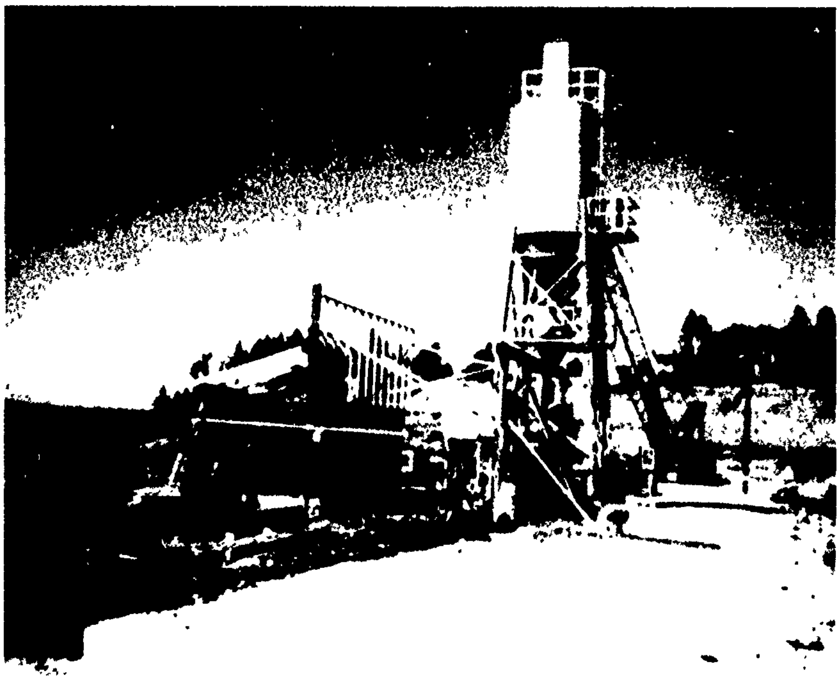


Looking toward spillway from left bank. TH-60 (left) and Schramm (right) air rotary drills installing primary grout (recompression) pipes on either side of the cutoff wall alignment through the deep section of the canyon.

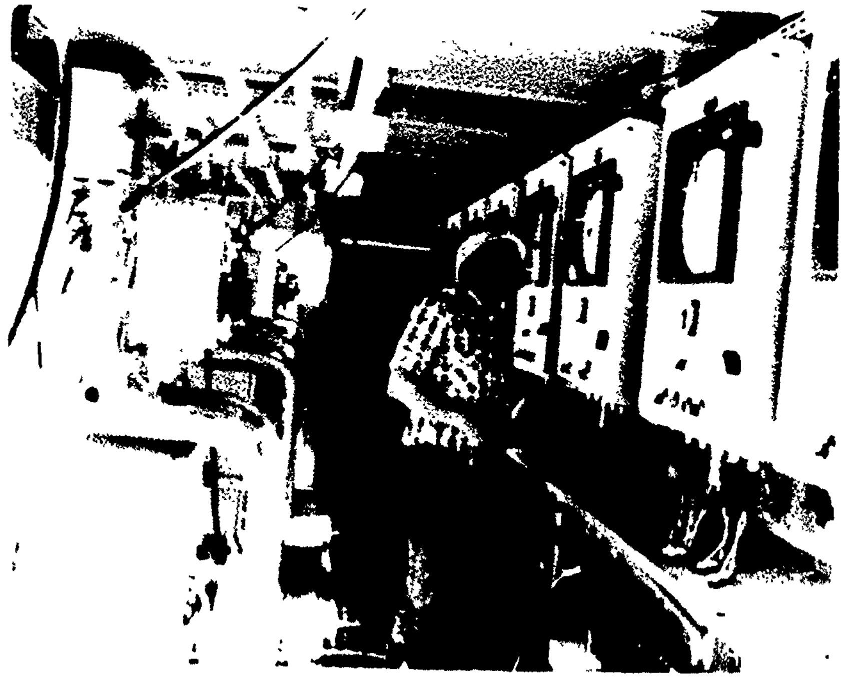


"ENPASOL" Drilling Parameter Recorder mounted on Schramm drill gathering information on soil conditions in core.

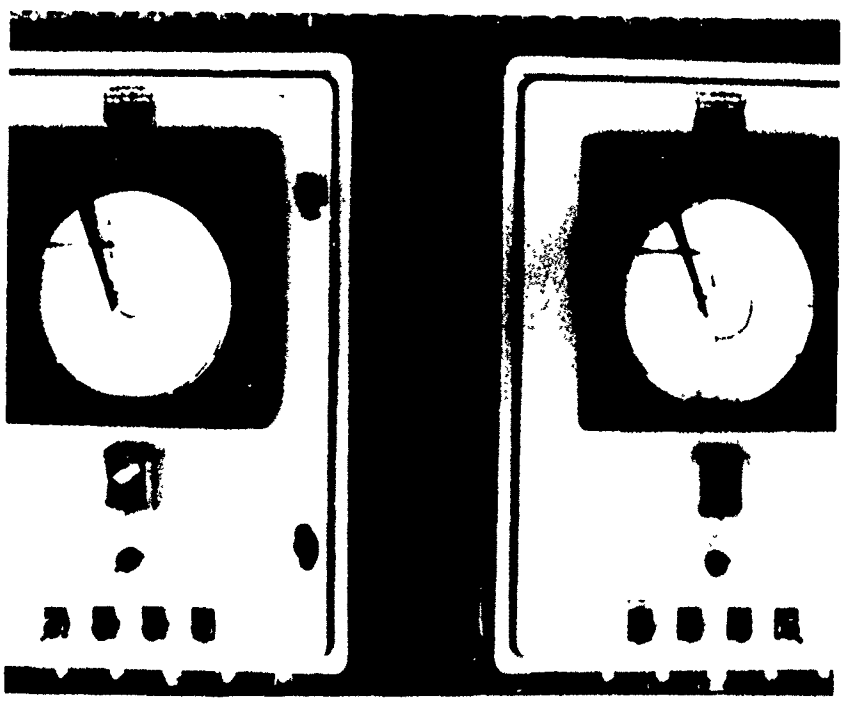
Recompression grouting plant. Eight 2x5 pumps which feed the grout to the holes. Mixing feed tanks on opposite side of structure. Grout recorders are inside. Looking upstream.



View of grout plant with ancillary 100 ton bulk cement hopper and 4 c.y. digester, below. Digester feeds mixing tanks (not installed yet) on right side of plant. Recorders and pumps on left.



Grout recording instrumentation. Grout pumps and motors to left.



Grout recorders. Records on 24 hour circular paper disk. Plots flow rate, volume and pressure against time.

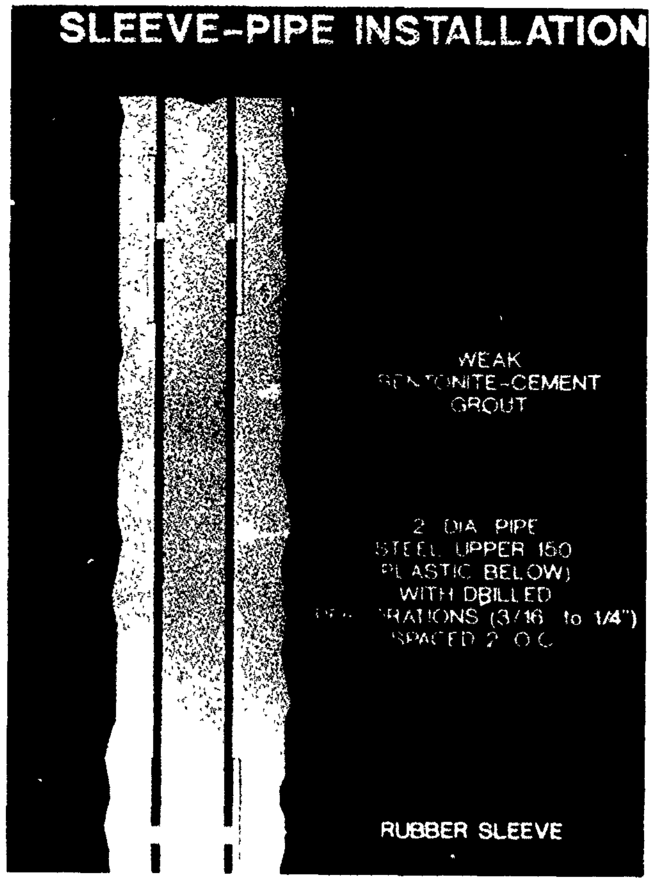
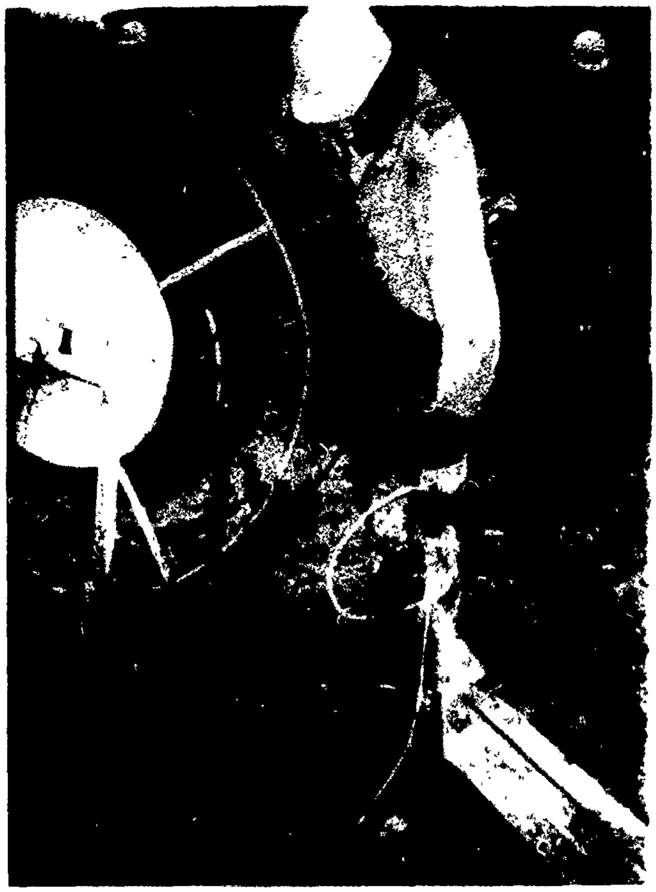


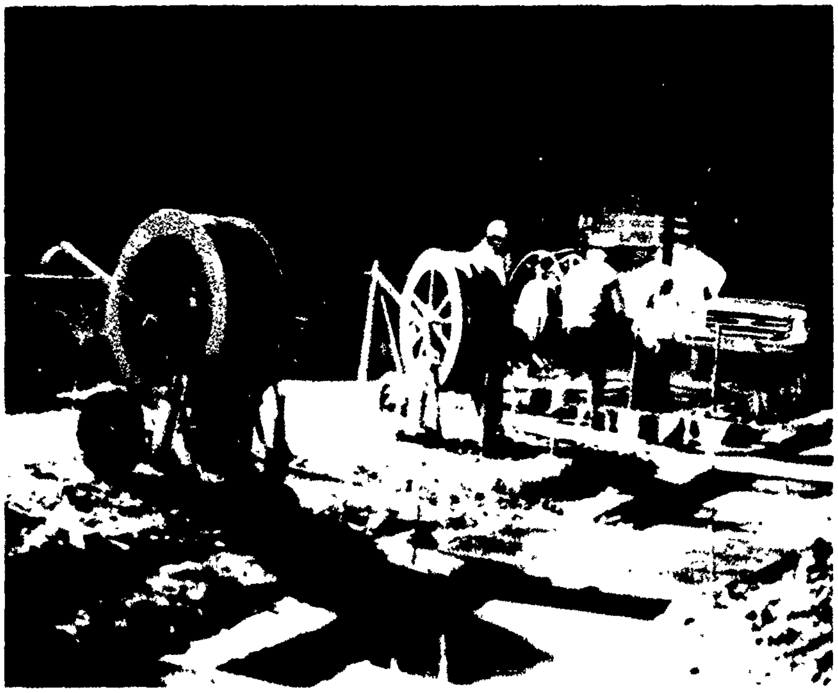
Diagram of sleeve grout pipe (tube-a-machette).

Two-inch sleeve pipe (bottom). Workman holding double-packer great injection pipe. Tube at right is packer pressure line.

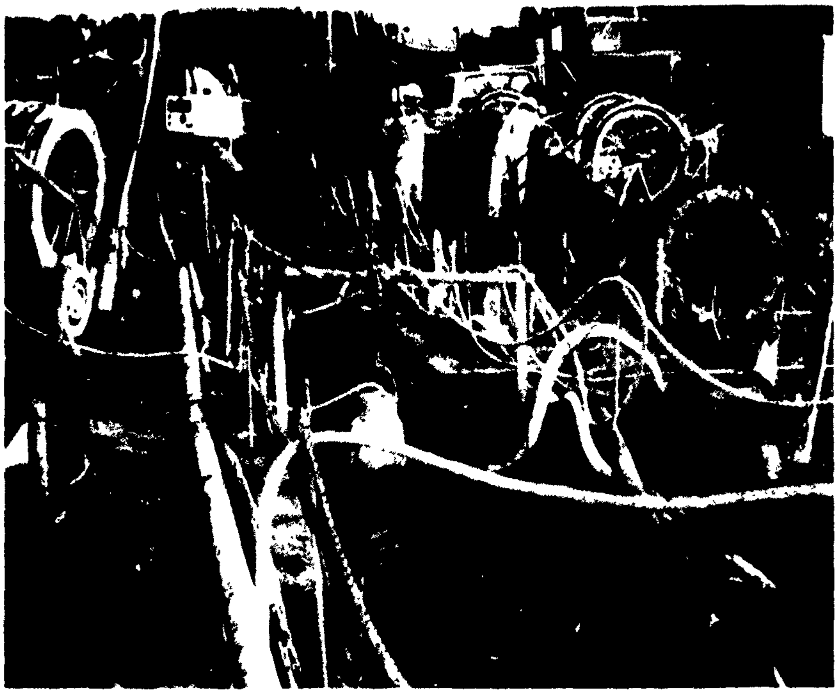


Double packer with grout feed line being fed down to target depth from the "Joseph" (mobile storage spool).

Hydraulic hand pump mounted on the base of the Joseph for inflating double packer.

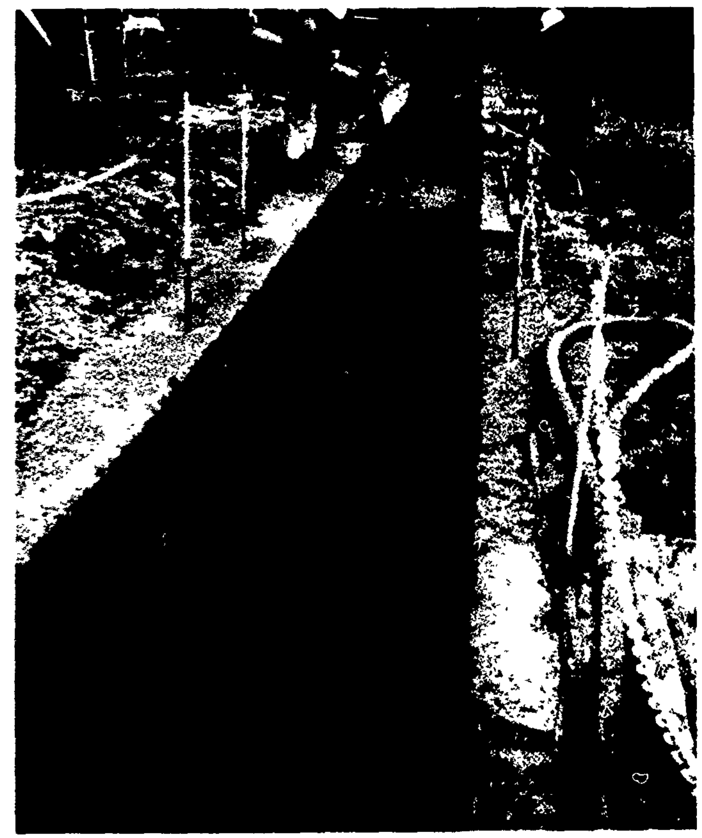


Night shift recompression grouting. Looking toward left bank.



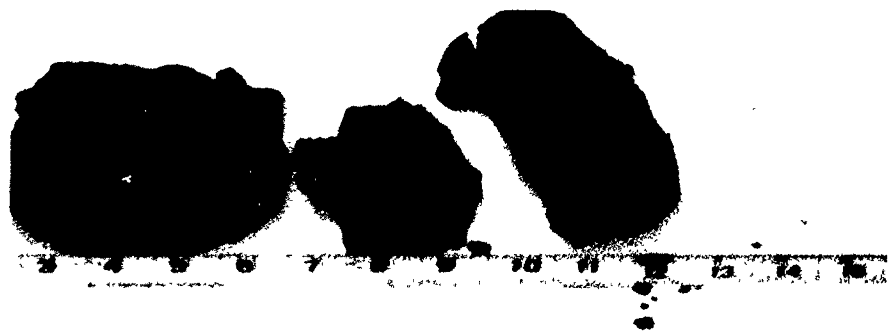
Worksite congestion during primary grouting operations. Note drill rigs in background drilling secondary grout holes. Looking toward left bank.

Piezometer P-55 during primary grouting operations. Note tube P-1 (middle) has grout extruding from end. P-55 is located upstream of the cutoff wall along the left bank, P-1 is deepest tube (400 ft. +).



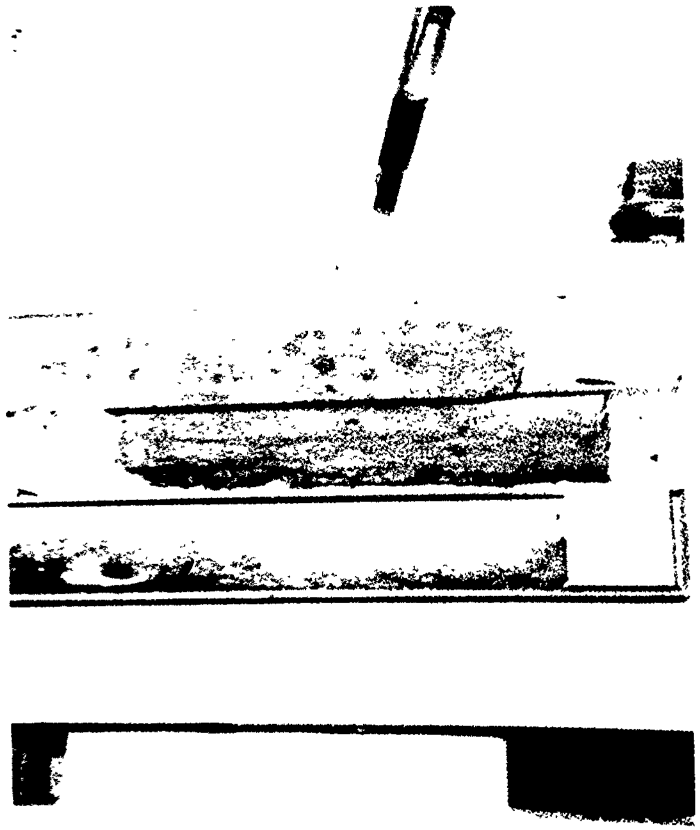
Concrete cap placed in upper 2-3 feet of unexcavated trench to contain grout. Upper portions of core sidewalls are re-stressed, reducing the potential of a platform wedge failure into an "open" panel.

Evidence of longitudinal hydrofracture. Bentonite stringers found in trench about station 15+30, prior to placing concrete cap. Down stream guide wall at left. Stringers all along cutoff wall alignment. Remnant filtrate cakes varied from 1/8 - 1/4 inch thick.



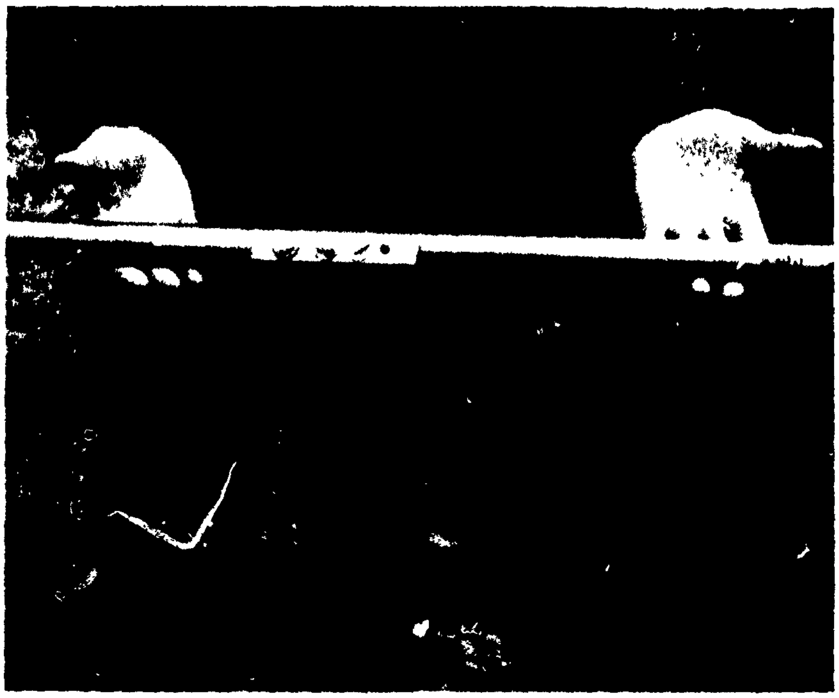
Core fragments from station 15+30 showing bentonite filtrate cake left from repeated hydrofrature occurrences. Note "double event" recorded at 8 inch mark.

Quality control concrete core (90-DD-QE127-181) showing the joint details between panels 126 and 127, at 120 ft. depth. High quality, tight joint had only film of bentonite between panels for the 20 ft. intersection cored.



Panel 126/127 boundary. Longitudinal striations are hydrofraise cutting pattern on Panel 126.

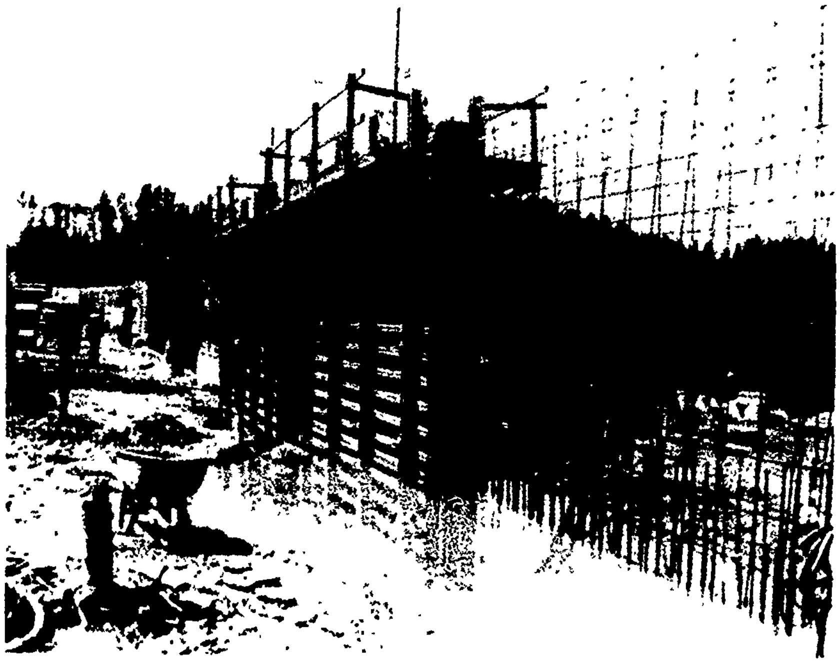
"Navidrill" directional drilling tool used to drill and straighten Q.C. concrete cores inside 32-40 inch wall up to 400 ft. deep.



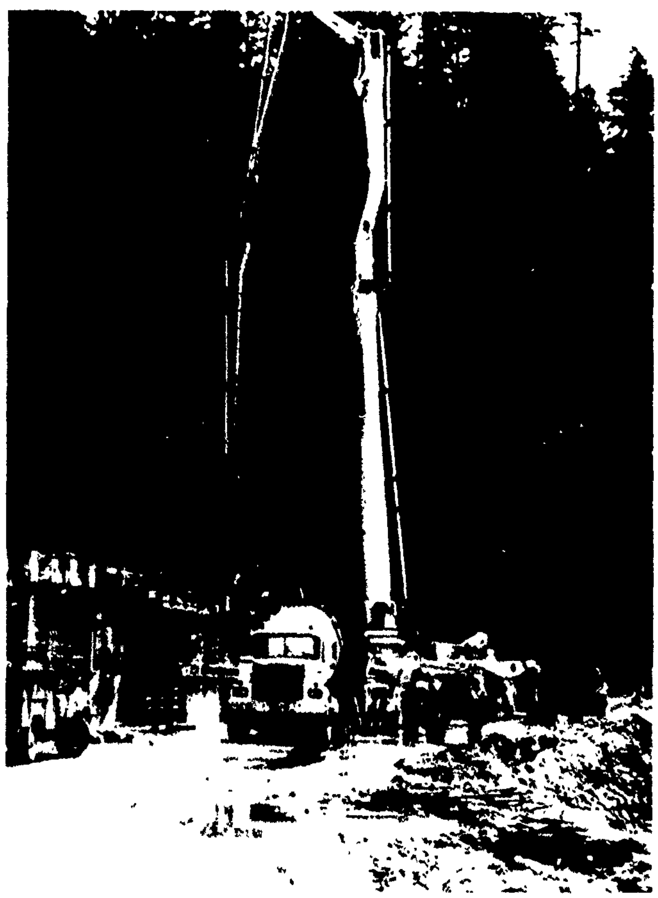
Navidrill recording device inside tool.

Driller checking "picture" taken with Navidrill
to determine core hole orientation.

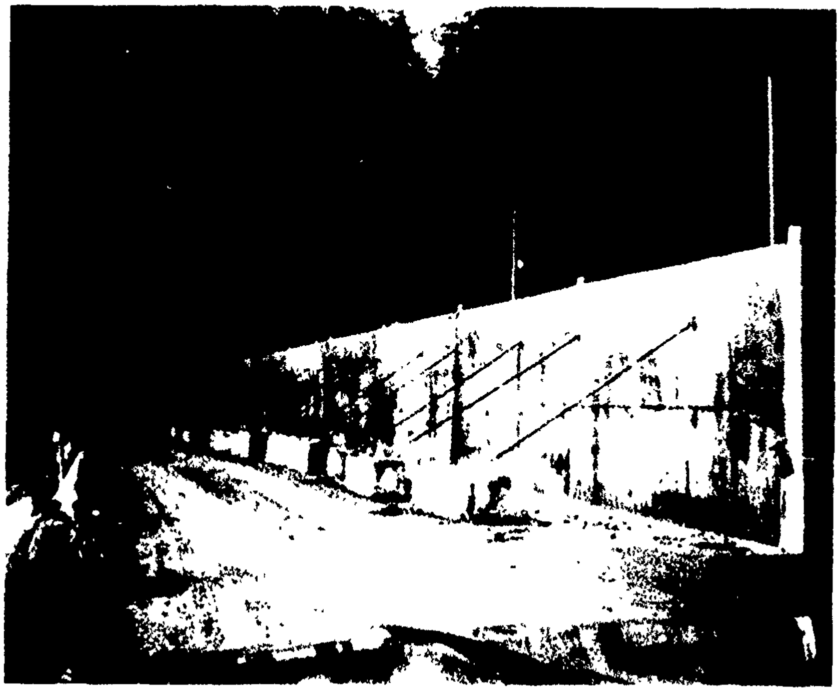
"American-fraise." Hydrofraise motor mounted on
D-9 dozer used to trim top of concrete panels for
good C.I.P. wall bonding surface.



C.I.P. wall rebar and forming in place.



Concrete pump truck used for C.I.P. wall placements.



C.I.P. wall braced off of ecology blocks after form stripping. Note inclinometer casings extended up through wall.

Backfill and compaction during dam raising. Thin strip either side of CIP wall was compacted with hand-operated tamper.